CAL/APT PROGRAM — ASPHALT TREATED PERMEABLE BASE (ATPB)

Laboratory Testing, Performance, Predictions, and Evaluation of the Experience of Caltrans and Other Agencies

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16. Abstract

This report is one of a series detailing the results of the CAL/APT program, a research effort being performed jointly by the University of California,Berkeley, Pavement Research Center and the California Department of Transportation (Caltrans). It contains an evaluation of the performance of asphalt treated permeable base (ATPB) in asphalt concrete pavements. The evaluation is based on observation, laboratory testing of ATPB, and computer simulation of representative pavement structures. The evaluation includes: 1) a summary of Caltrans experience with ATPB and drainage systems relative to their development and implementation, together with observations of field performance with respect to maintainability and stripping of asphalt treated materials; 2) a summary of the characteristics and performance of ATPB materials and drainage systems used by two other highway agencies; 3) results of laboratory investigation of the stiffness and permanent deformation characteristics of ATPB mixes, including the effects of soaking and loading while saturated; and 4) the results of analyses of representative pavement structures to quantify the expected effects of as-compacted and soaked ATPB on pavement performance

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FINANCIAL DISCLOSURE STATEMENT

This research has been funded by the Division of New Technology and Research of the State of California Department of Transportation (contract No. RTA-65W485). At the time of the study described in this report, the total contract was to extend for a five-year period (1 July 1994 through 30 June 1999) in the amount of $5,751,159. This contract was later amended to extend to 30 June 2000 in the amount of $12,804,824. This report presents an evaluation of the performance of asphalt treated permeable base (ATPB) in asphalt concrete pavements based on observations, laboratory testing of ATPB, and computer simulations of representative pavement structures.

The results are intended to supplement the results of other laboratory tests, analyses, and HVS tests on drained and undrained pavement structures being performed as part of Goal 1 of the CAL/APT program Strategic Plan. The CAL/APT program Strategic Plan is concerned with validation of existing Caltrans design procedures for both new and overlaid pavements.

IMPLEMENTATION STATEMENT

Results of this study suggest that the current practice in which ATPB is placed directly under the asphalt concrete layer should be reconsidered. The argument for placing ATPB in this location is that the ATPB layer intercepts water that enters through the pavement surface before it can reach the unbound layers and cause damage. The original philosophy considered the ATPB layer to intercept subsurface water as well, however surface water is the primary concern in the current Caltrans philosophy.

Two reasons for water to enter the pavement through the surface are cracks in the surface and/or a poorly compacted and therefore permeable asphalt concrete layer. By 1) reducing the permeability of the asphalt concrete through improved hot mix compaction, and 2) incorporating sufficient thickness of this layer to mitigate the potential for load associated cracking (using analytically-based methodology of the type being developed as part of the CAL/APT program), the reason for placing ATPB under the asphalt concrete layer could be eliminated. Moreover, the use of the "rich bottom concept," in which the fatigue response of the lower portion of the asphalt concrete layer is enhanced, would also lead to reduced water permeability of the asphalt concrete layer.

For those applications where the use of ATPB is still planned, its mix design should be modified to improve its water resistance. This can be done with increased binder content and modified binders such as rubberized asphalt. In addition, properly designed soil or geotextile filters should be placed adjacent to the ATPB layer in the pavement structure to prevent the ATPB layer from becoming clogged. Finally, to insure continued efficacy of the ATPB, effective maintenance practices for edge and transverse drains should be established and distributed to the Caltrans Districts. If these recommendations are followed, then the gravel factor for ATPB should be increased to 2.0.

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EXECUTIVE SUMMARY

This report is one of a series detailing the results of the CAL/APT program, a research effort being performed jointly by the University of California, Berkeley Pavement Research Center and the California Department of Transportation (Caltrans). It contains an evaluation of the performance of asphalt treated permeable base (ATPB) in asphalt concrete pavements based on observation, laboratory testing of ATPB, and computer simulation of representative pavement structures. The evaluation includes: 1) a summary of Caltrans experience with ATPB and drainage systems relative to their development and implementation, together with observations of field performance with respect to maintainability and stripping of asphalt treated materials; 2) a summary of the characteristics and performance of ATPB materials and drainage systems used by two other highway agencies; 3) results of laboratory investigation of the stiffness and permanent deformation characteristics of ATPB mixes, including the effects of soaking and loading while saturated on these parameters; and 4) the results of analyses of representative pavement structures to quantify the expected effects of as-compacted and soaked ATPB on pavement performance.

The objectives of the studies reported herein were as follows:

1. obtain an indication of the effects of water on the stiffness and permanent deformation characteristics of ATPB through laboratory testing;
2. relate the soaking performed in the laboratory to field conditions, including the effects of soaking on stiffness and permanent deformation;
3. understand the structural effects on pavement performance of pavement structures with ATPB including the effects of soaking the ATPB – this phase is intended to

provide a bridge between HVS test results on pavements containing ATPB in a relatively dry state and field performance of pavements in which the ATPB will likely be subjected to soaking;

4. understand the design philosophy that has led to the use of ATPB in Caltrans pavements and review the implementation of that philosophy to date; and
5. evaluate this information in order to make recommendations for the use of ATPB in flexible pavements in California and ways to insure that it is properly drained.

The results included in this report are intended to supplement the results of other laboratory tests, analyses, and HVS tests on drained and undrained pavement structures being performed as part of Goal 1 of the CAL/APT program Strategic Plan, which is concerned with the validation of existing Caltrans design procedures for both new and overlaid pavements.

Chapter 2 provides a summary of Caltrans experience with ATPB. It includes a review of the development of the use of ATPB, observations of field performance problems recently documented by Caltrans, and the results of a survey of selected Districts to determine maintenance issues associated with ATPB and edge drain systems.

The original philosophy of Caltrans was to include asphalt treated permeable materials in certain pavements with the goal of more quickly removing both surface and subsurface water entering the pavement section, and thereby maintain the structural capacity of the unbound materials in the pavement. In practice, Caltrans has included ATPB as a part of the structural cross-section of asphalt concrete pavements for more than 15 years. During this period some problems have been reported. Stripping of asphalt from the aggregate has been observed in some ATPB materials; this phenomenon has also been reported by other agencies using similar materials. Maintenance of edge drains has been a problem for some Caltrans districts, particularly where the drains have been added as retrofits rather than as design features in new or reconstructed pavements. In addition, several districts have reported frequent clogging of their drainage systems. The current trend is for reduced maintenance funding and staffing, which has resulted in diminishing ability to regularly maintain the drainage systems; this trend is expected to continue. These observations stress the importance of examining ATPB in a soaked state.

Chapter 3 summarizes the experience of the Indiana Department of Transportation (IDOT) and the Ontario Ministry of Transportation (OMOT), Canada with ATPB.

While the extent to which an ATPB might remain saturated has not been extensively examined, results of the study by IDOT suggest that the ATPB can remain saturated for a significant period of time after a rain event. These results indicate that it is important to study the response of saturated ATPB and that loading representative of moving traffic be applied to the material in this condition to fully evaluate its expected performance in the field.

Experience of both IDOT and OMOT emphasizes the need for a filter layer to prevent the migration of fines into the drainage layer from untreated layers. The OMOT study also reported that stripping of the asphalt binder in the ATPB was observed, especially in areas below cracks. Similar observations were reported by Caltrans personnel with respect to some ATPB installations.

Chapter 4 reports the results of laboratory tests of ATPB. Because 1) the field experiences of Caltrans (Chapter 2), and IDOT and OMOT (Chapter 3) include reports of saturated ATPB, and 2) the HVS test on the pavement section containing the ATPB was performed with this material in the "dry" state (Section 500 RF), a series of laboratory tests were performed to evaluate the effects of water on ATPB. The ATPB tested was designed following the current Caltrans specifications and collected during construction of the HVS pavement and a field pavement section in Contra Costa County (Vasco Road). The laboratory-compacted specimens prepared from these materials were tested in the as-compacted state and after soaking in water for up to ten days at 20°C. Permanent deformation tests using repeated load triaxial compression tests were also performed on specimens in both the as-compacted and saturated states. The results of the laboratory tests showed significant reductions in resilient modulus and increased permanent deformation rates after soaking, as well as loss of cohesion and binder stripping at particle interfaces when subjected to repeated loading while saturated.

Chapter 5 contains the results of analyses of pavement structures containing ATPB and conventional aggregate base and designed according to the Caltrans design procedure. The goal of these analyses was to obtain comparative performances of the two pavement structures in terms of fatigue and permanent deformation as measured by subgrade strain. Both the ATPB (drained) and aggregate base (undrained) pavements were designed using NEWCON 90 for a range of Traffic Indexes (8 to 14) and subgrade R-values of 5, 20, and 40. Performance of the drained pavements was simulated using ATPB stiffnesses obtained from the laboratory tests on the ATPB in the as-compacted state and after ten days of soaking.

The results of the simulations indicated that the drained pavements would have substantially longer service lives than would the undrained pavements. Performance of the drained pavements containing ATPB that had been soaked for ten days was poorer than that of pavements in which the ATPB was in the as-compacted state, however, the performance of the drained pavements in both conditions still exceeded the performance of the undrained pavements.

Chapter 6 includes the conclusions and recommendations of the study. From the information presented in this report, it is recommended that Caltrans reconsider its policy on the use ATPB in pavement sections. In this regard, the location of the ATPB will likely influence any decision. For example, if seepage is occurring into the pavement section from the subgrade, then the use of ATPB is warranted. However, for this application, mix design requirements for the ATPB should be modified. In addition, a suitable filter layer based on reliable filter design principles should be incorporated.

Current practice in which the ATPB is placed directly under the asphalt concrete layer should be reconsidered. The argument for placing ATPB in this location is that it can intercept water that enters through the pavement surface. The two major reasons that water may enter the pavement structure through the pavement surface are the existence of cracks in the surface and/or a permeable asphalt concrete layer. The need for ATPB in this location could be eliminated by reducing the permeability of the asphalt concrete, which can be achieved through changes to mix design, improved compaction and construction practices, and by mitigating the potential for load associated cracking through improved compaction and incorporation of sufficient asphalt concrete thicknesses. Asphalt concrete thicknesses sufficient to reduce the probability of fatigue cracking can be designed using analytically-based methodology of the type discussed herein.

The value of the ATPB as a structural layer when it has no role as a drainage layer does not justify its use. Data reported herein suggest stiffness values on the order of 1×10^6 kPa (150,000 psi) for ATPB material in the dry state. At the same temperature, conventional asphalt concrete will have stiffness values in the range 5.5 to 7×10^6 kPa (800,000 to 1,000,000 psi). If the ATPB is saturated and the material is sensitive to water, the stiffness may be reduced to the value of about 5×10^5 (75,000 psi), about one-half that in the dry state. While the stiffness values in both the dry and wet state appear larger than representative stiffness values for untreated granular bases, improved pavement performance could, from a cost standpoint, be better achieved by proper mix design and thickness selection (e.g., use of the rich bottom concept) and through improved construction practices, particularly improved compaction of the asphalt concrete.

If a decision is made to continue the use of ATPB directly beneath the asphalt concrete layer, then the actions recommended above for the use of ATPB in pavements with subsurface seepage should be undertaken for its use beneath the asphalt concrete layer as well. These include the following.

1. *Development of requirements to improve water resistance of ATPB Materials.* The resistance of ATPB materials to water damage (stripping, loss of cohesion, and stiffness loss) can be significantly reduced by changes in the specifications for ATPB. The most likely variables for change are increased asphalt content and changes in binder specification including the use of asphalt-rubber. Any changes in the ATPB specification would need to ensure sufficient permeability and constructability comparable to materials currently in use.

2. *Define methods to maintain the drainage capacity of ATPB layers.*

   If ATPBs are to remain effective as drainage layers, it will be necessary to insure that:

   1. adequate filter layers are provided adjacent to the ATPB to minimize the intrusion of fines;

   2. edge and transverse drains are maintained to prevent their filling with fines or becoming clogged.

The current practice of using a heavy prime coat on the aggregate base as the filter material should be evaluated to ascertain its effectiveness. Guidelines should be developed for proper design of filters using either soils or geotextiles. Recommended maintenance practices for edge and transverse drains should be established and distributed to the Districts, and adequate equipment and staffing to follow these practices should be provided.

If the above recommendations are followed to improve the resistance of ATPB to the action of water, then its gravel factor should be increased to a value of 2.0.

1.0 INTRODUCTION

Caltrans has used asphalt treated permeable base (ATPB) as part of its structural pavement sections for more than 15 years. The advent of the CAL/APT program has provided Caltrans an opportunity to determine the effectiveness, from a structural standpoint, of using ATPB in pavement sections.

In Caltrans nomenclature, pavements that include a layer of ATPB are referred to as “drained” pavements, while those that contain only conventional aggregate base are referred to as “undrained” pavements. As a part of Goal 1 of the CAL/APT Strategic Plan, Heavy Vehicle Simulator (HVS) tests have been conducted at the Richmond Field Station to evaluate the performance of drained and undrained asphalt concrete pavement test sections in an “essentially dry” condition; that is, no water was applied to the pavement surface and the ground water table remained at a depth of about 4 m during the test program. In addition to the HVS tests, laboratory repeated load tests were performed on specimens of the ATPB in both the dry and saturated conditions and an assessment of the performance of the pavements in both the “dry” and “wet” conditions was made using multilayer elastic analysis.

This report addresses 1) the design philosophy that led to the incorporation of ATPB into Caltrans flexible pavement designs, 2) its implementation by Caltrans, 3) analyses of its effects on fatigue and subgrade rutting performance in the dry state and when subjected to wet conditions, and 4) a critical assessment of the current Caltrans usage of ATPB.

1.1 Purpose

The drained pavement sections containing ATPB were tested in the dry state in the HVS tests. The results included in this report are intended to address questions regarding the performance of drained pavement structures when subjected to wet conditions. The approach described in this report was intended to facilitate the interpretation of the results of the HVS tests, completed as a part of Goal 1, in terms of actual field conditions.

1.1.1 Objectives

The objectives of the research described in this report include the following:

1. measure the effects of water on the stiffness of ATPB through laboratory testing,
2. relate the soaking performed in the laboratory and its effects on mix stiffness to field conditions,
3. understand the structural effects of inclusion of an ATPB layer in the pavement on pavement performance and the effects of soaking the ATPB on that performance in order to provide a bridge between HVS test results on pavements containing ATPB maintained in a relatively dry state and field performance for pavements where the ATPB will likely be subjected to soaking,
4. understand the design philosophy that has led to the use of ATPB in Caltrans pavements and review the implementation of that philosophy to date, and
5. from an assessment of this information, provide recommendations pertaining to the use of ATPB in flexible pavements in California.

1.1.2 Hypothesis – Effects of Wet Conditions in the Field Performance of ATPB

Comparative performance of drained and undrained pavement test sections observed during HVS loading with "dry" conditions has indicated that ATPB can improve the performance of flexible pavements.

On the other hand, field observations of ATPB in "wet" conditions have indicated that there is a significant probability of stripping of the asphalt from the aggregate. If this occurs, it is hypothesized that the propensity for fatigue cracking will increase because of the higher strains in the asphalt concrete layer caused by reduced stiffness of the ATPB. The reduced stiffness of the ATPB could also result in increased vertical strains in the underlying untreated materials leading, in turn, to potential increases in rutting at the pavement surface.

This report explores this hypothesis through evaluations of Caltrans experience as well as the experience of other agencies with ATPB, laboratory triaxial repeated load, stiffness testing of "dry" and "wet" ATPB specimens, and analyses of representative pavement structures using the results of the laboratory tests and multilayer analysis. The hypothesis is supported by the HVS tests in the pavements containing both the ATPB and the conventional aggregate base.

1.2 Organization of Report

Chapter 2 of this report summarizes and details the implementation of the design philosophy that led Caltrans to include ATPB in pavement structures. The chapter also summarizes the results of a project undertaken to evaluate the stripping of ATPB in Caltrans pavements and the results of a brief survey of district maintenance staff regarding maintenance practices and problems with edge drains and ATPB.

Chapter 3 summarizes work performed by other organizations regarding the amount of soaking found in their pavements containing materials similar to the ATPB used in Caltrans pavements. Also discussed are the types and amount of water damage to the ATPB-like materials observed in the field by those organizations.

Chapter 4 presents the results of laboratory testing of ATPB materials in dry and wet conditions. The laboratory tests included measurements of resilient modulus (stiffness) of as-compacted ATPB, after soaking at 20°C, and after repeated loading while saturated at 20°C.

Chapter 5 presents the results of simulations conducted to examine the predicted fatigue and subgrade rutting performance of Caltrans drained pavements containing as-compacted ATPB, ATPB after soaking, and Caltrans undrained pavements. The simulations incorporated the results of the laboratory testing on as-compacted and soaked ATPB.

Chapter 6 presents the conclusions drawn from the results and provides a critical assessment of current Caltrans practice relative to the use of ATPB.

2.0 REVIEW OF CALTRANS EXPERIENCE WITH ASPHALT TREATED PERMEABLE BASE

Caltrans has experimented with asphalt treated permeable materials for use as drainage layers in highway pavements for at least 35 years and has used asphalt treated permeable base (ATPB) as a standard component in new pavement designs since 1983.

Local government agencies in California and other state departments of transportation have followed Caltrans' lead in the use of ATPB. Examples include Contra Costa County and the Washington State DOT. Local government agencies in California typically use the same specifications and pavement structural design procedures developed by Caltrans.

This chapter presents a review of the development of current Caltrans methods for ATPB and its use, observations of field performance problems recently documented by Caltrans, and the results of a survey to determine maintenance issues associated with edge drain systems and ATPB. The findings from the review provide important information regarding improved specifications for ATPB materials and the incorporation of ATPB into the Caltrans procedure for flexible pavement thickness design.

2.1 Drainage Layer Design

The philosophy that led to development of criteria incorporated in current Caltrans practices for the use of ATPB as a drainage layer appears to be based on work reported by Lovering and Cedergren in 1962. *(1)* Their work attempted to mitigate, through improved drainage, the problems associated with larger pavement water contents leading to strength reduction in pavement materials, and the presence of free water permitting pumping of unbound soils particles from the pavement under traffic loading.

While Caltrans currently uses ATPB in a manner that follows the recommendations of Lovering and Cedergren in many ways, there are some significant differences between current practice and the original recommendations.

2.1.1 Recommendations by Lovering and Cedergren

The factors to be considered for design of effective pavement drainage systems, as presented by Lovering and Cedergren, include:

1. expected permeability of pavement surface and probable precipitation;
2. permeability of the water bearing soil and probable hydraulic head;
3. drainage gradients, both transverse and longitudinal, available for removal of water;
4. permeabilities of the various elements of the drainage system;
5. proper grading of drainage aggregate and transition filters to prevent clogging; and
6. provision of outlets as required for the capacity of the drainage layers.

Lovering and Cedergren recognized that drainage should be provided to take care of water infiltration into the pavement from the subgrade and from the pavement surface.

Infiltration of water from the subgrade typically depends upon the permeability of the subgrade material. A subgrade with a low fines content may have high permeability and allow a great deal of water to enter the pavement. A subgrade with a high fines content may have a low permeability and so allow little water to enter the pavement from below, but may be a source of fines that can infiltrate the base and subbase layers to reduce their permeability to a level insufficient for adequate drainage of the pavement.

Infiltration of water from the pavement surface depends upon the ability of water to pass through the asphalt concrete or portland cement concrete surface. The presence of unsealed cracks or joints in the surface significantly increases the infiltration rate of water into the underlying layers.

Even when uncracked, the infiltration rate through the surface of flexible pavements can vary considerably depending upon the permeability of the compacted asphalt concrete. Recent research by Allen *et al.* substantiates the observations cited by Lovering and Cedergren that good compaction of the asphalt concrete surface is required to reduce surface infiltration. *(2)* As can be seen in Figure 2.1, by reducing construction air-void contents from about 10 percent to about 5 percent, the permeability of the pavement surface can be reduced by approximately two orders of magnitude. *(2)* Lovering's and Cedergren's observation that surface infiltration rates can often exceed those from the subgrade is probably still applicable today, considering that asphalt concrete air-void contents of 8 percent and greater are common under current Caltrans compaction specifications, which require 95 or 96 percent density relative to laboratory test maximum density (California Test Method 304). *(3)*

Lovering and Cedergren proposed the use of an open-graded drain rock layer directly beneath the asphalt concrete surface for flexible pavements constructed on subgrades containing few fines. Treatment of the open-graded drain rock with asphalt to improve constructability was recommended because untreated drain rock will not provide a stable platform for the operation of construction vehicles. For subgrades containing a large percentage of fines, a filter type aggregate drainage layer was suggested as long as it provided enough drainage capacity to account for both surface and subsurface infiltration.

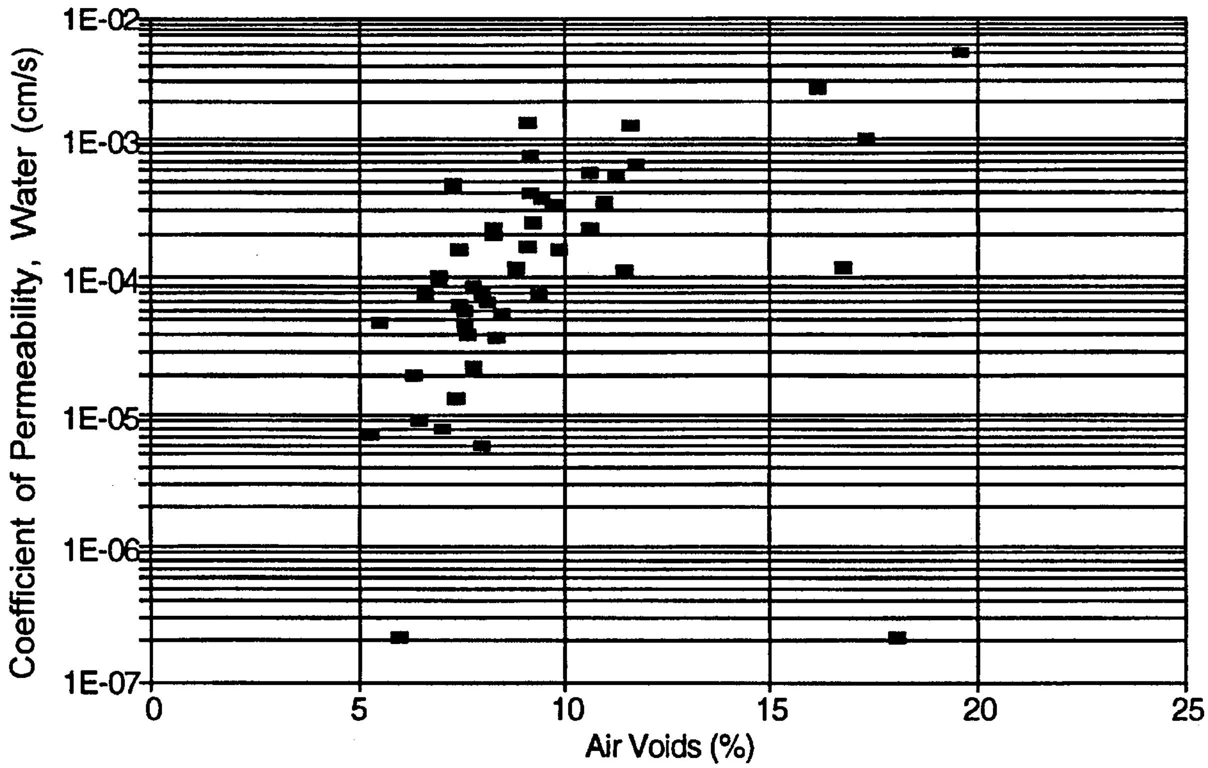

Figure 2.1. Variation in the coefficient of water permeability with air-voids (from SHRP Report A-396 [2]).

A two-layer drainage system was suggested for other cases, such as pavements with high levels of surface infiltration and subgrades with large percentages of fine particles, or pavements with high subsurface and/or surface infiltration rates and subgrades with sufficient fine particles to clog a filter type drainage layer. The two-layer drainage system consisted of a filter type aggregate layer with an open-graded drain rock layer on top of it and just below the asphalt concrete surface.

Lovering and Cedergren recommended the use of a single-sized material treated with 2 to 3 percent asphalt for the open-graded drain material. This material was expected to provide high flow capacity and acceptable stability. The asphalt was intended to provide stability for construction. Laboratory test results cited for materials treated with 2 percent asphalt included the permeabilities shown in Table 2.1.

Table 2.1 Asphalt treated permeable materials recommended by Lovering and Cedergren.

Particle Sizes (mm)	Permeability (m/day)	Proportion of Crushed Particles
25-38	36,600	10 percent
9.5-19	10,700	50 percent
2.36-4.75	1,800	100 percent

Analyses performed by Lovering and Cedergren indicated that layers of these materials of 75-mm thickness or somewhat thicker would be sufficient to accommodate many cases of surface and sub-surface infiltration, provided that an adequate filter was used when necessary to prevent infiltration of fines from the underlying unbound layers. A 100-mm thick ATPB material (25-mm to 38-mm particle sizes) with a 100-mm thick filter below was indicated to be sufficient to drain most water bearing soils with ample allowance for partial clogging on the upper and lower boundaries.

Lovering and Cedergren emphasized that for the drainage layers to be effective, sufficient drainage outlets with adequate flow capacity had to be provided at the edge of the pavement. They also emphasized that the drainage outlets have to be maintained so that they do not become clogged and create a "bathtub" effect in the drainage layer.

Lovering and Cedergren assigned no structural value to the thin drainage layer. They did indicate that if the thickness and permeability of aggregate base required for the structural design provided adequate drainage, then the use of an asphalt treated permeable drainage layer would probably be uneconomical. Aggregate base was considered to be a filter type material provided that it had a sufficiently low percentage of fine particles.

2.1.2 Current Caltrans Drainage Design Using ATPB

Caltrans currently uses one thickness and one type of asphalt treated permeable material in its pavement designs, adopted originally in 1983. The material was fairly uniform with 80 to 100 percent of the particle sizes between the 9.5 mm and 19 mm sieve sizes. At the time the material was adopted, it was required that 25 percent of the particles be crushed. The asphalt content was typically specified to be 1.5 percent by mass of aggregate, and AR-4000 asphalt was used. Measured permeability was about 4,575 m/day (15,000 ft/day).

These requirements were similar to those for the 19 mm material proposed by Lovering and Cedergren. Primary differences being in asphalt content (of 2 to 3 percent for Lovering and Cedergren versus 1.5 percent for Caltrans) and in aggregate (50 percent crushed for Lovering and Cedergren versus 25 percent for Caltrans). Neither the very coarse (38-mm maximum aggregate size) nor the fine (4.75-mm maximum aggregate size) materials were included in the Caltrans standard specifications (Section 29). *(4)*

Deformation problems experienced during construction using the mixes conforming to the initial specifications led to changes in 1984. These changes included an increase to 90 percent crushed particles, and the use of an AR-8000 asphalt rather than the AR-4000 grade. In addition, allowable maximum asphalt content was increased to the range of 2.0 to 2.5 percent and the material designation was changed from asphalt treated permeable material to asphalt treated permeable base (ATPB).

The specification was amended again in 1987 to require that the asphalt be added to the mix when the aggregate temperature was between 135°C and 163°C (275°F and 325°F). The previous specification had required that the aggregate temperature for mixing be below 135°C. The purpose of the 1987 change was to increase the stiffness of the binder during the mixing process. This increase in stiffness was desired to eliminate the instability (also referred to as "tenderness") encountered in the ATPB after placement. *(5)*

ATPB is currently used in a 75-mm thick layer, whereas Lovering and Cedergren had advocated adjusting the thickness of the drainage layer and the material used depending upon the drainage requirements for the pavement section. Where fines were present in the unbound soil layers in amounts sufficient to risk clogging of the ATPB, Lovering and Cedergren recommended the use of an underlying base material with a gradation that would permit it to serve as a filter. Current Caltrans practice assumes that the Class 2 aggregate base and prime coat between the aggregate base and the ATPB serve as the filter. *(6)*

Caltrans typically constructs an edge drain system to discharge the water flowing to the edge of the pavement, and a system of collector pipes and outlets to carry the water away from the pavement. In the 1980s, many pavements were retrofitted with edge drains, the majority of which were for portland cement concrete pavement. There is no record of retrofitting existing pavements with ATPB.

2.2 Development of Caltrans Gravel Factor for ATPB

Lovering and Cedergren assumed that the asphalt treated drainage material would not contribute to the structural capacity of the pavement for thickness design purposes. They assumed that the improvement in the structural capacity would come from improved drainage, rather than the stiffness of the drainage material.

During the implementation of ATPB into the Caltrans standard procedures, the decision was made to include the ATPB in thickness designs as a structural material. It was therefore necessary to assign a gravel factor (G_f) for use in the Caltrans thickness design procedures. It was thought reasonable that the gravel factor for an asphalt treated base material would have a value between that of a Class 2 aggregate base (G_f=1.1), and dense graded asphalt concrete (G_f=1.46 to 2.54 depending upon the traffic index). Based on engineering judgment, G_f of 1.4 was assumed.

To validate this assumed gravel factor, a section of rural two-lane highway with a traffic index of about 8 (State Route 36 in Tehama County) was reconstructed with a test section containing the structures shown in Table 2.2. *(7)*

Table 2.2 Comparative Pavement Structures on State Route 36 Used to Validate ATPB Gravel Factor in 1980. *(7)*

	Undrained Section	**Drained Section**
Asphalt Concrete	122 mm (0.4 ft.)	122 mm (0.4 ft.)
Base	503 mm (1.65 ft.) Aggregate base	75 mm (0.25 ft.) ATPB 427 mm (1.4 ft.) Aggregate base
Subgrade	R-value = 17	R-value = 19

The ATPB material used in the drained test section had a similar gradation to the current specification. The asphalt content was 3.6 percent rather than the 2.5 percent currently used.

Deflections were measured on the two sections three months after construction in November 1979, again in April 1980 after one winter, and in January 1981 during the second winter. These deflection measurements are summarized in Table 2.3.

Table 2.3 Average Deflection Measurements on State Route 36 Used to Validate Gravel Factor for ATPB. *(7)*

Date (months after construction)	**Undrained Section Deflection (inches)**	**Drained Section Deflection (inches)**	**Percent Difference**
Nov 79 (3)	.013 westbound .011 eastbound	.008 westbound .007 eastbound	38 36
Apr 80 (9)	.023 westbound .023 eastbound	.017 westbound .015 eastbound	26 35
Jan 81 (14)	.013 westbound .012 eastbound	.011 westbound .011 eastbound	15 8

It can be seen that average deflections are smaller in the section that contains asphalt treated permeable material. The percent difference in deflections determined from the November 1979 measurements was used with the Caltrans overlay design procedure to back-calculate a gravel factor of 1.4 for the asphalt treated permeable material for each direction.

A sensitivity analysis in which the deflections on the sections containing ATPB were decreased by 0.001 in. resulted in a back-calculated G_f of 2.4. A variance of ± 0.001 in. or more is typical of Caltrans deflection measurements. Considering that this small change in deflection resulted in a gravel factor in the same range as asphalt concrete – thought to be unreasonable – it was concluded that the gravel factor validation was inconclusive. While a research plan was proposed for 1981 to further evaluate the structural value of ATPB, the initial G_f of 1.4 for ATPB has remained unchanged to this date.

It is interesting to note that the difference in deflections between the drained and undrained sections on the State Route 36 test section decreased significantly after two winters in service, suggesting the possibility of some deterioration of the stiffness of the ATPB material after exposure to water.

2.3 Observed Field Performance of Caltrans ATPB

By the early 1990s, Caltrans personnel suspected that ATPB materials were experiencing stripping in at least some projects, including both asphalt concrete and portland cement concrete pavements. Stripping was considered to be a contributor to faulting in portland cement concrete pavements and loss of structural strength in asphalt concrete pavements. A research project was performed to investigate the extent of stripping and potential causes. *(5)*

Nine in-service pavement sections containing ATPB or asphalt treated permeable material layers were cored and the drainage layer material was classified by visual observation as being either "stripped" or "nonstripped." The extent of intrusion of fines into the drainage layer was also observed. In addition, the asphalt from each specimen was extracted and tested for viscosity at 60°C, penetration, and asphalt content; the asphalt source was also identified. Seven of the nine pavements had asphalt concrete surfaces and two had portland cement concrete surfaces. The extent of observed stripping and fines intrusion is summarized in Table 2.4.

These results indicate that stripping of the ATPB within 10 years of construction is not uncommon in asphalt concrete surfaced pavements. Stripping may be rapid and common at locations where large quantities of water enter the ATPB layer from the surface, such as at the joints of portland cement concrete pavements and at cracks in asphalt concrete pavements or

Table 2.4 Summary of ATPB Stripping Performance. [from Wells *(5)*]

Project	Stripping Observed?	Fines Intrusion Observed?	Years in Service	Other Observations
Asphalt Concrete Pavements				
1	No	Yes	3	
2	No	No	3	
3	Yes	No	10	Stripping in lower 25 mm of ATPB layer only
4	No	No	13	State Route 36 validation section
5	Yes	No	1	High porosity of asphalt concrete surface, stripping in bottom of ATPB only
6	Yes	Yes	7	
7	Partial	No	3	Particles separated, asphalt intact on particles
Portland Cement Concrete Pavements				
1	Yes	No	2	Severe stripping at joints where water enters
2	Yes	Yes	4	Severe stripping at joints where water enters

when an asphalt concrete surface has high porosity due to poor compaction. In those pavements where stripping occurred, it had often proceeded to the point at which no asphalt was found on the aggregate particles or anywhere else in the ATPB layer.

Fines intrusion, while not as prevalent as stripping, occurred in three of the nine pavements. There was no indication that the ATPB layers were completely clogged with fines.

Statistical analysis of the correlation between observed stripping and the factors of pavement age, recovered penetration, recovered viscosity at 60°C, and asphalt content indicated that asphalt content was the only highly significant factor. Larger asphalt contents resulted in reduced likelihood of stripping. Although not statistically significant, trends were observed that indicated mixes with more rapid aging of the binder (smaller recovered penetrations and larger recovered viscosity) had a greater tendency towards stripping. It was also found that the Caltrans stripping test (CTM 302) did not appear to identify ATPB mixes that would strip.

Recommendations resulting from the project report are as follows:

- Until more definitive research can be performed, the asphalt content of ATPB should be increased from 2.5 to 3.0 percent by mass of aggregate.
- A performance-related quantitative test procedure should be developed to measure the stripping potential of ATPB mixes.

2.4 Survey of Caltrans Districts Regarding Drainage Performance

Caltrans maintenance staff face different environments, traffic, pavements, and resource conditions among the districts. A brief, non-statistical survey was made of the districts to obtain a qualitative understanding of the experience of districts in maintaining pavements constructed with asphalt treated permeable base layers and/or edge drains.

Wherever possible, the district maintenance person interviewed was a maintenance supervisor or someone with "front-line" experience and an understanding of pavement behavior.

2.4.1 Questions Asked

Questions asked in the survey were as follows:

1. Approximately what centerline mileage of ATPB and/or edge drain equipped pavement is there in the district or your portion of the district?

2. About how much of that drained pavement is asphalt concrete pavement (ACP) and how much is portland cement concrete pavement (PCCP)?
3. About how long have those features been installed?
4. What kind of maintenance, if any, is performed on the drainage systems?
5. How often is maintenance performed, and is it on a routine or emergency basis?
6. Is that maintenance effective?
7. Have you observed clogging of the drainage systems with fine materials from unbound soils layers or the surface?
8. Have you seen stripping of the ATPB, as observed from cores or asphalt in the edge drains and outlets?

Note that in some cases, the questions were not applicable to the district or the respondent did not have the requested information.

2.5 Survey Results

The results from the districts surveyed have been categorized in terms of rural or urban and by rainfall intensity (low and high). Urban areas often have more constraints on traffic closure than rural areas, affecting their ability to perform maintenance. Pavement performance in areas with greater rainfall is assumed to be more dependent upon the presence of drainage systems than areas with lesser rainfall. The districts, shown in Figure 2.2, were somewhat arbitrarily grouped as follows:

Rural/High Rainfall—Districts 1, 2, 3 and 5

Rural/Low Rainfall—Districts 6, 8 and 9

Urban/High Rainfall—District 4

Urban/Low Rainfall—Districts 11 and 12

Responses were not obtained from Districts 7 and 10 during the very short time frame in which the survey was conducted.

2.5.1 Rural/High Rainfall Districts

2.5.1.1 District 1

District 1 has had pavements constructed with edge drains and ATPB for about 10 years. About 40 centerline km of US Route 101 include ATPB. The edge drains have probably not been maintained by contract since they were constructed. However, the District now has under consideration flushing of the drains by contract, and is investigating the required equipment and procedures to do this. At times, maintenance forces have observed discharges containing fine-grained soil, and have often found vegetation growing in the pipes, conditions observed by removing the inspection plugs.

In general the surfaces of the ATPB equipped pavements appear in good condition. No special problems have been observed in the pavements with edge drains or ATPB.

The District has used the film stripping test (CTM 302) to evaluate aggregates used in ATPB. They believe that the test has provided them useful information for aggregate from at least one quarry with potential ATPB stripping problems.

Figure 2.2. Map of Caltrans districts (from California State of the Pavement Report, 1995, Caltrans Maintenance, Sacramento).

2.5.1.2 District 2

District 2 has about 275 centerline km of PCCP and about 150 centerline km of ACP with edge drains. The only pavement with ATPB is on State Route 36, the section used to develop the gravel factor for ATPB and described earlier.

The District has access to a "vactor" truck, which is used to clean out edge drains every one to two years. Clogging of the edge drains with fine grained soils has been observed regularly. Fines have been observed in the discharges from and in deposits at the outlets. Complete clogging of some edge drains has occurred; however, most do not reach this stage. The clogged drains are often discovered when slabs are replaced.

Edge drains in PCCP have caused a great deal of trouble for maintenance forces. District personnel expressed the opinion that equipping PCCP constructed on cement treated base (CTB) with edge drains was not an effective strategy. CTB is not free draining and water entering the pavement through surface cracks does not easily get to the drains. Material from the CTB may also contribute to clogging of the edge drains. Pressure grouting of PCCP has resulted in the clogging of drains with grout. Currently maintenance forces run water continuously through the drains when grouting.

The opinion that edge drains may accelerate damage on those PCCP sections where surface cracking allows water to enter the pavement was expressed. Little or no difference in performance was noted between PCCP sections with and without edge drains when there was no surface cracking. ACP sections equipped with edge drains have caused few problems for the District maintenance forces. Little difference in performance was noted between ACP sections with and without edge drains.

2.5.1.3 District 3

District 3 has edge drains primarily on PCCP sections of Interstate 5, and State Route 113, which also has some ATPB. Many of these drains are retrofits. The maintenance person contacted expressed the opinion that edge drains were "not maintainable" and contribute to early pavement failures. The district has a vactor truck used for maintenance of pavements with edge drains; this maintenance of edge drains is regularly performed, usually in response to clogged drains.

It was observed that in addition to cleaning the drains, the vactor truck had a tendency to pump water under the PCCP slabs, essentially negating the purpose of edge drains. The district has tried cleaning the drains with air, but the process was not effective. Other maintenance operations performed in the District include digging out the edge drains, rebuilding them, and repairing edge failures where the drains are located. Drain outlets are marked and easy to find.

Clogging of edge drains is regularly observed and is not a localized problem. Fine-grained soils are observed pumping through the slab joints and cracks as well. Fabrics have been ineffective in keeping edge drains from clogging. Outlets as well as edge drains are found to be completely clogged with fine-grained soils; it is believed that these blocked edge drains have contributed to early failure in the PCCP sections. Hydraulic pressures under PCCP sections have been observed to force crack sealants from the pavement surface.

2.5.1.4 District 5

District 5 has edge drains on about five to ten percent of pavement sections. At most, one or two projects have ATPB. The oldest projects have been in place about ten years. Until recently the district was continuing to retrofit pavements with edge drains, and is still including edge drains and ATPB in new construction. Edge drains are cleaned almost every year by shooting water through them; however, the district does not have a vactor truck. Maintenance staff have not assessed the effectiveness of the current routine maintenance. They are currently placing markers at each outlet so that they can easily be found.

There is not much evidence of clogging of the drains. There is no evidence of ATPB stripping, in part because there is so little ATPB in the district. There have not been any major recurring problems with drainage systems.

The overall effectiveness of edge drains was a subject of divided opinion. One maintenance manager thought that they enhanced pavement performance, while the other had no faith that they were effective. It was noted that edge drain retrofits add considerable work to rehabilitation projects.

2.5.2 Rural/Low Rainfall Districts.

2.5.2.1 District 6

District 6 (250 mm of rainfall per year) has been including edge drains in their crack/seat/overlay projects on Interstate 5 for the past three years. On those projects, fabric is placed from edge to edge as a filter material. Edge drain retrofits were performed in the district in the late 1980s and early 1990s. There is no knowledge of ATPB in the district.

No routine maintenance is performed on the edge drains. Maintenance forces focus their attention on keeping the surface sealed as a means of limiting water intrusion into the pavement. At times, drains do clog and the district uses its vactor truck to clean them.

Maintenance has had a great deal of difficulty in cleaning clogged drains because the outlets were connected to the edge drains using "T-connectors" (90° turns) and the jet router cannot make the turn. If there is a large quantity of material in the drain, it can take days to clean. From the time of construction, it has taken three winters to clog a drain.

In one case, an edge drain was rehabilitated and was found to be completely clogged with fine-grained soil. However, it was also found that filter fabric had only been placed on the top of the edge drain and not surrounding it.

2.5.2.2 District 8

District 8 has about 320 centerline km of PCCP and about 32 centerline km of ACP that are equipped with edge drains. Most of the PCCP was equipped with edge drains in the early to mid-1980s. The edge drains on ACP are newer. ATPB has been included in some newer pavements.

No routine maintenance is performed on edge drains. There is almost never any need to perform maintenance of any kind on the drains.

There have been no specific problems associated with edge drains. Water is seldom seen coming out of the outlets. The effectiveness of pavement drainage is hard to determine because rain events are so short in duration that it is difficult to determine whether water at the edge of the pavements has come from the outlets or from surface runoff. There was doubt about whether the retrofit edge drains provide any benefit to pavement performance.

2.5.2.3 District 9

District 9 has about 65 centerline km of pavement with edge drains only and about 160 centerline km of pavement with ATPB. Only ACP sections constructed in the past seven years have ATPB. The PCCP sections equipped with edge drains are older.

Most edge drains are never maintained, or have been maintained only once. There is no special equipment in the district for this activity, and the district must either borrow a vactor truck from a neighboring district or rent one. It was noted that maintenance of drainage systems is not specifically included in design evaluations or budgeting.

Clogging of edge drains does not appear to be a problem. However, it was noted that one section appeared to have a problem with water freezing at the drain outlet and blocking it, while water within the pavement remained unfrozen or even in a vapor state. It was thought that this may have led to stripping of the ATPB in the section. Edge drain rehabilitation is typically not included in rehabilitation projects.

Other problems included difficulty in keeping plastic outlet pipes at grade because they degrade in the sun and curl up, and because they are snagged by maintenance equipment.

It was observed that in some places the drainage systems have continuously flowing water. However, because rainfall in the district is typically less than about 150 mm per year it was thought that drainage layers and edge drains were not required for the majority of conditions.

2.5.3 Urban/High Rainfall Districts

2.5.3.1 District 4

District 4 has about 630 centerline km of pavements with edge grains, including both ACP and PCCP. They also have about 135 centerline km of ATPB. Routine maintenance is seldom performed on edge drains. When a problem is observed, a vactor truck is used to clean the drain. A previously used hydro-auger was preferred for this work. When a problem is suspected during rain events, outlets are inspected to see if they are flowing water. However, there is no routine inspection of outlets.

Clogging was primarily observed in pavements where the edge drains were not equipped with filter fabric. Newer sections with filter fabric have much less tendency to clog. A major problem with pavements that have been retrofitted with edge drains is the 100 mm wide trench at the pavement edge. It has a tendency to quickly develop cracking and potholes because it is very near where the majority of truck loadings pass and because it intercepts surface runoff.

The interviewees indicated that they thought that inclusion of drainage layers and edge drains in new pavements improves pavement performance. They observed that edge drains often continuously flow full when they intercept groundwater. However, they suggested that retrofit edge drains resulted in poorer pavement performance and were not worth the cost and effort.

2.5.4 Urban/Low Rainfall Districts

2.5.4.1 District 11

District 11, in its western sub-district (rainfall about 230 to 290 mm per year), has edge drains on about 100 centerline km of PCCP on Interstate Routes 5 and 8. The oldest edge drains were installed in the late 1970s and early 1980s and the newest were installed about five years prior to the interview (approximately 1992). There was no knowledge of any sections with ATPB.

Edge drains are flushed approximately every three years using the district's vactor truck. Maintenance forces often have trouble finding outlets. It was noted that edge drains have a tendency to fill with fine dust that hardens with time if the drains are not flushed occasionally. Pavements that are retrofitted with edge drains have experienced acute failures at saw cuts in the PCCP slabs.

2.5.4.2 District 12

District 12 has some pavements with edge drains, but was not sure of any pavements with ATPB. Many of the edge drains are being installed as new lanes are added to existing routes. Most of the edge drains have been installed within the past ten years.

Edge drains are cleaned about every two to three years. Clogging of the drains is a problem if they are not maintained on this schedule. In recent years, the district has rented a vactor truck to perform this work. Most of the material found in the drains was thought to be entering from the pavement surface rather than from the underlying soil layers. Most clogging was found in the edge drain pipes and not in the outlets.

The interviewees suggested that edge drain maintenance technology should be improved. Edge drains in the district need regular maintenance and often require lane closures if outlets are to be reached in the median. Lane closures are a major problem for maintenance staff because they are costly and dangerous. Another problem is that environmental regulations in the district are requiring that material recovered from the drains must be tested to see if it contains hazardous waste. If it is found to be safe, it can be disposed of; if it is found to be hazardous it must be placed in a sanitary landfill, which is costly.

In some locations, it was noted that edge drains run water continuously and must constantly be maintained to prevent clogging.

2.6 Findings

The original recommendation to include drainage systems in Caltrans pavements, by Lovering and Cedergren, emphasized designing the drainage system for the water infiltration and potential clogging conditions of each project. Options for asphalt treated permeable base (ATPB) materials, drainage layer thicknesses, and filters were included in the original recommendations. They also suggested that it was not necessary to include a drainage system where conditions did not warrant one. Routine maintenance to preserve the benefits of the drainage system was emphasized. For design purposes, the drainage layer (ATPB) was assumed not to contribute to the structural capacity of the pavement.

Caltrans currently uses one ATPB material and one thickness for all applications, including locations with low rainfall and little groundwater infiltration. The asphalt content of the standard ATPB material is typically lower than that originally proposed by Lovering and Cedergen based on their laboratory tests.

For the asphalt contents currently used, stripping of asphalt from the ATPB aggregate appears to be a common problem, sometimes to the point that there is no asphalt remaining in the material. This is particularly a problem where large flows of water enter the pavement through the surface, such as at joints in PCCP and joints and cracks in ACP. Larger asphalt contents appear to reduce or eliminate stripping with the current aggregate gradation. The current film stripping test (CCTM 302) appears to be inadequate to identify which mixes will strip, although one district has used it successfully on at least one occasion.

Maintenance staff have widely varying schedules for the routine cleaning of edge drains, ranging from not at all to once per year. All use a vactor truck, either their own or a rented or borrowed one, to clean edge drains. The need for maintenance also appears to vary widely. Three of the four rural districts with high rainfall regularly clean their drains, as do both urban districts with low rainfall. Three out of three rural districts with low rainfall and the single urban district with high rainfall do not routinely clean edge drains. When maintenance is routinely performed, it is usually every one to three years. It appears that drain cleaning is largely performed in urban districts in Southern California, and in the rural districts with high rainfall.

Three of ten districts surveyed reported regular occurrences of edge drain clogging. Those districts did not all fit into the same urban/rural or high/low rainfall categories, and the existence of drain clogging probably depends upon local soils and surface conditions.

Most maintenance forces interviewed had in common three observations:

1. Edge drain retrofits do not work well, especially in comparison to pavements where they are part of the new pavement design,
2. The ability to maintain edge drains should be considered more in design (outlet geometry, access for equipment, outlet location marking) and budgeting than it is currently.
3. Designers should better consider whether drainage will work and whether it is needed for different projects.

3.0 REVIEW OF THE USE OF ASPHALT TREATED PERMEABLE BASE IN OTHER AGENCIES

3.1 Introduction

In addition to its use by Caltrans, ATPB is being used as a drainage layer by a number of highway agencies both in California and elsewhere. The experiences of some of those agencies have been documented in the literature. These experiences were compared with Caltrans practice to identify common problems, problems unique to California, and potential improvements to California practice. In addition, there was minimal information available within Caltrans to calibrate the water conditioning procedures used for the laboratory testing with field conditions (to be described in Chapter 4 of this report) and it was hoped that the experiences elsewhere would provide some additional guidance.

Two studies reported in the literature were evaluated; the first by the Indiana DOT (IDOT) and the second from Ontario, Canada. *(8, 9)* The study by IDOT allowed a comparison of their materials used in drainage layers, especially aggregate grading and asphalt content, with Caltrans requirements. The IDOT report highlighted the need to prevent the migration of fines from the untreated layers into the drainage layer to ensure continued satisfactory performance of the drainage system. The study also highlighted the fact that the major source of water infiltration into the Indiana pavements is through surface cracks. The report contained information from which guidelines for the water conditioning of laboratory test specimens were obtained.

The Ontario, Canada, study also permitted specifications used by the Ontario Ministry of Transportation (OMOT) and Caltrans for drainage layer materials to be compared. The Ontario experience, like that of Indiana, emphasized the need for a filter layer to prevent the migration of fines into the drainage layer from the untreated layers. The study reported that stripping of the asphalt binder was observed, especially in areas below surface cracks, as was also experienced in some ATPBs in California in similar locations (Chapter 2).

This chapter discusses these studies and their findings and provides some comparisons between this information and California conditions.

3.2 Materials and Pavement Structure Comparison

To facilitate comparison between the conditions in Indiana, Canada, and California, the pavement structures used in the three investigations are shown in Table 3.1, and the material specifications for the drainage layers are shown in Table 3.2.

For the Indiana pavements, the No. 5C and No. 2 permeable bases are treated and used as drainage layers. The No. 5D treated base and No. 53 untreated base are used as both filters and a structural base between the drainage layers and the subgrade.

The average of the gradation limits for the drainage layers used in Indiana, Canada, and California are plotted in Figure 3.1.

The No. 5C permeable base and the No. 5D filter base materials used in the Indiana study met all specifications. The No. 2 permeable base was marginally out of specification. For the Canadian study, no data were reported for field sampling of the materials. It was reported that the ATPB met Caltrans requirements. *(10)*

Table 3.1 Pavement Structures.

Pavement Layer	**Pavement Layer Thickness (mm)**		
Indiana Pavement Sections	**Section 1**	**Section 2**	**Section 3**
Asphalt Concrete No. 11 Surface	25	25	25
Asphalt Concrete Binder	(No. 9) 76	(No. 8) 76	(No. 8) 76
Permeable Base No. 5C	76	76	—
Permeable Base No. 2	228	228	304
Filter base	(No. 5D treated)	(No. 53 untreated) 216	(No. 53 untreated) 216
Canada Pavement Sections	**Section 1**	**Section 2**	**Section 3**
Surfacing	203 (plain jointed PCC slab)	25 (Asphalt Concrete, friction coarse) 30 (Asphalt Concrete)	80 (Asphalt Concrete)
Base (unreinforced concrete)		230	
Permeable Base	100 (treated)	110 (treated)	100 (treated)
Base (primed and sanded)	75 (Granular A) 380 (Granular C)		
Typical Caltrans ATPB Pavement Sections at University of California, Berkeley Richmond Field Station			
Asphalt Concrete Surface	150		
ATPB	76		
Aggregate Base	183		
Aggregate Subbase	230		

The gradation data show that the Caltrans ATPB has a more uniform grading than the Indiana materials and slightly less than the Ontario material. This will result in the Caltrans material having a greater permeability than the Indiana materials and about the same as the Ontario material.

Table 3.2 Material Properties.

	Indiana		Canada		Caltrans	
	Permeable Base No. 5C	**Permeable Base No. 2**	**Filter (untreated) No. 53**	**Filter (treated) No. 5D**	**Open graded drainage**	**ATPB**
Asphalt Content (Percent by weight of aggregate)						
	3.1-4.7	2.6–3.6	-	4.2-5.4	2.0-3.1	2.0-3.0
Sieve Size (mm)	**Gradation, Percent Passing**					
37.5	100	30-60	100	100	-	-
25.4	70-98	20-50	80-100	80-99	100	100
19	50-85	15-40	70-90	68-90	90-100	90-100
12.7	28-62	10-35	55-80	54-76	-	35-65
9.5	15-50	15±5	-	45-67	20-55	20-45
#4	15±5	3-20	35-60	40±5	0-10	0-10
#8	3-20	2-15	25-50	20-45	-	0-5
#16	10-15	1-10	-	12-36	-	-
#30	1-10	0-7	12-30	7-28	-	-
#50	0-7	0-6	-	3-18	-	-
#100	0-6	0-4	-	1-12	-	-
#200	0-4	-	5-10	0-5	-	0-2

The asphalt content of the Caltrans ATPB is lower than that of the Indiana drainage layers and approximately the same as reported from the Ontario study.

3.3 Rainfall Comparison

The purpose of the drainage layers is to quickly drain water that penetrates the pavement in order to prevent accelerated pavement damage that can occur when water stands in a

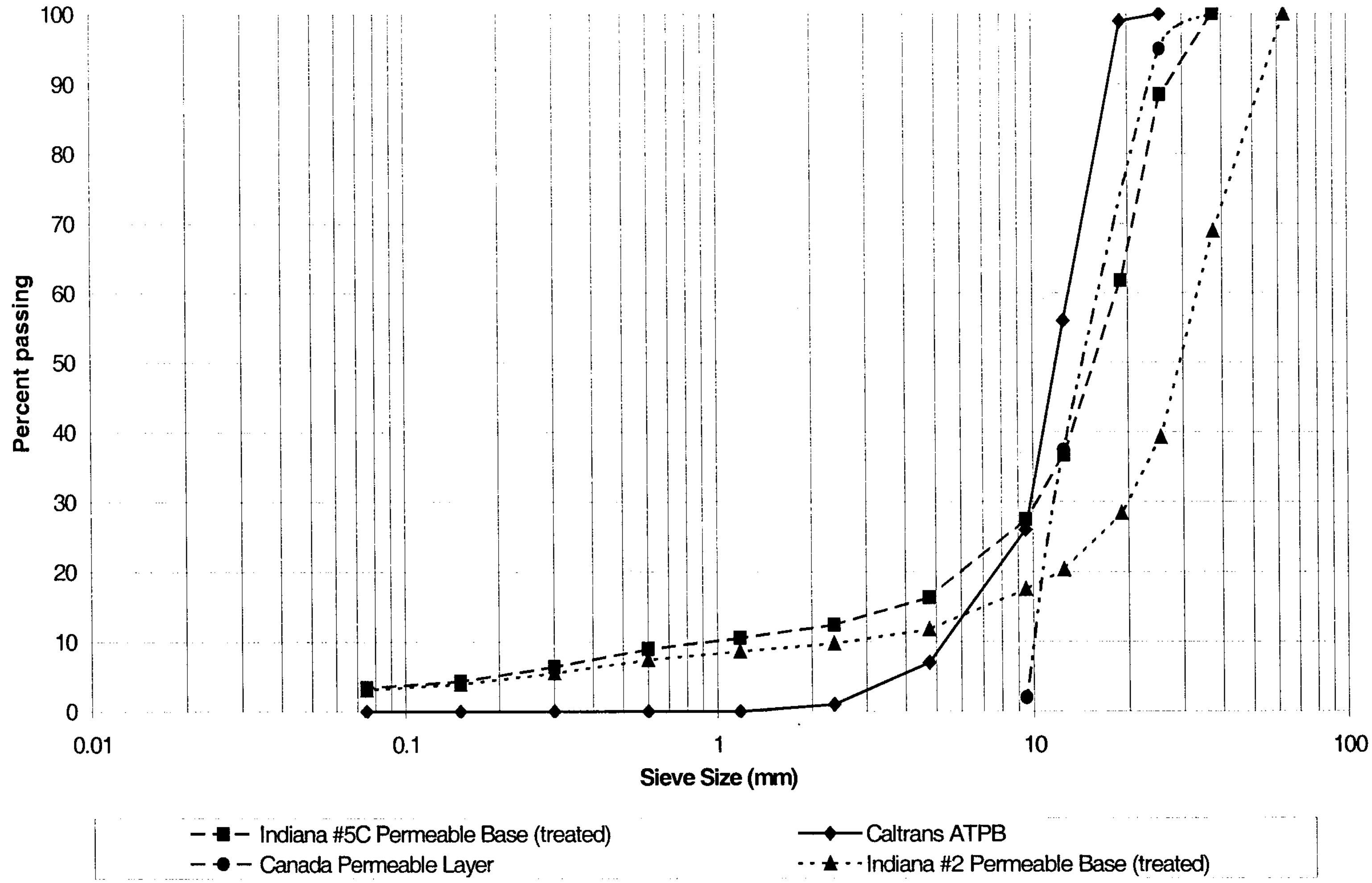

Figure 3.1. Comparison of Drainage Layer Gradings (Average of Upper and Lower Specification Limits).

pavement. During a heavy rainfall of short duration, a greater percentage of the rain will run off the surface of the pavement and not infiltrate, whereas if the rainfall is slow and steady more rain will probably infiltrate the pavement.

Rainfall in Indiana occurs periodically in all seasons and consists typically of thunderstorms of relatively short duration, during which large amounts of precipitation occurs. In Ontario, Canada, the rainfall may either occur as thunderstorms of short duration or more consistent steady precipitation that may last for a few days. In both Indiana and Ontario, rain occurs throughout the year and snow is experienced during the winter.

Rainfall in most of California typically consists of long, consistent rain that may last for many days, especially in the northern part of the state. Thunderstorms are more typical of the southern Central Valley and in the deserts; however, there is less annual rainfall in California than that which occurs in either Indiana or Ontario. The rainy season is largely confined to the period November through March. *(11,12)*

It is expected that the slow, consistent rainfall in California, particularly in the northern part of the state, could result in the greater infiltration into the pavement through the surface.

3.4 Case Studies of Pavements Containing ATPB

Outflow from the pavement drainage system is perhaps the best available measure of the efficiency of the drainage system; it is a function of both the amount of infiltration into the pavement and the permeability of the drainage layer. If the drainage layer becomes infiltrated with fines that fill the voids and reduce the permeability, the amount of outflow will be reduced.

A measure of outflow that is often used is the ratio of outflow to rainfall volume, and is termed drainage efficiency. Where subgrade seepage is minimal, this ratio more appropriately represents the pavement condition in combination with the drainage efficiency. A problem with this measure is that the amount of outflow is also dependent on the amount of rainfall infiltrating the pavement; this is not apparent from the ratio. Therefore, care should be taken when making comparisons of outflows from pavements in different rainfall regions.

The time required for the water that has infiltrated the pavement to pass through the drainage layer and into the drainage outlet system is another concern with drainage efficiency as it has been defined. If the capacity of the drainage layer and the outlet system is inadequate, water will be trapped in the pavement; this will lead to the same problems that occur when the drainage system is blocked.

This section presents summaries of drainage efficiency, saturation periods, and pavement performance for the Indiana and Ontario studies. An attempt is made to compare the conditions and results from these studies with California experience. In particular, the studies provide an opportunity to estimate the periods of time during which the ATPB in California may be saturated and the resulting effects on performance.

3.4.1 Indiana Study

The Indiana study included three field test sections with the pavement structures shown in Table 3.1. Data were reported regarding outflow from the pavements for specific rainfall events, saturation of the pavement layers, and migration of fines into the drainage layers. Each of the three sections had the same edge drain system.

During several rainfalls, rainfall amount and outflow were measured. Degrees of saturation of some of the pavement layers were measured prior to the outflow monitoring period. Comparisons were made between the ratios of the outflow and rainfall volume, and times to drain for the three test sections. (The time to drain is defined as the time from the end of the rainfall event until the outflow stops or becomes asymptotic.)

3.4.1.1 Rainfall

Three rainfall events were recorded and monitored. The amount of rainfall differed for the three sections, as did the periods of rainfall. Rainfall events lasted up to three days and the maximum accumulated rainfall in one event was 110 mm.

3.4.1.2 Saturation

Degrees of saturation measured prior to the monitored rainfalls ranged from 0.55 to 0.95 for the No. 5C permeable bases (Sections 1 and 2). The No. 2 permeable base layers (all three sections) were essentially saturated prior to the measured rainfall events. The filter layers exhibited degrees of saturation ranging from 0.25 to 0.45; the subgrades were not fully saturated prior to the rainfall events. Degrees of saturation in the permeable bases during and after the rainfall event were not presented nor were times that the permeable bases were saturated during and after a rainfall event.

3.4.1.3 Outflow

Outflows from the three sections were monitored during the rainfall events and while the outflows continued. Generally, the maximum outflow occurs during a storm if its duration is long, or shortly after the rain subsides for shorter rainfalls. Outflow will continue for a fairly long period after the rainfall, with the duration and intensity of the outflow depending on the rainfall and the capacity of the pavement drainage system.

A summary of the percentage outflow volume to rainfall volume resulting from the three events for the three test sections is shown in Table 3.3. For each event, the percentage outflow volume to rainfall volume was comparable for the three sections. However, the time to drain was considerably shorter for Section 1 than for Sections 2 and 3.

Table 3.3 Summary of Rainfall Events for the Indiana Sections.

Event	Section	Section 1	Section 2	Section 3
1	Percentage outflow volume to rainfall volume	-[a]	0.092	0.10
	Time to Drain (hours)	-[a]	38	38
2	Percentage outflow volume to rainfall volume	0.17	0.189	0.19
	Time to Drain (hours)	10.5	25.5[b]	25.5[b]
3	Percentage outflow volume to rainfall volume	0.18	0.15	0.155
	Time to Drain (hours)	13.5	47	47.25

[a]Data not recorded
[b]Flow continues with another rainfall at that point

Inspection of Section 1 found it to have the least amount of migrated fines silting up the drainage layer and the outlet drains. This is most likely due to the No. 5D filter preventing the migration of the fines from the underlying untreated materials. The filters of Sections 2 and 3 were composed of untreated materials. In this instance, it would appear that the untreated filters were not as effective in retarding the migration of fines as was the treated filter.

It should also be noted that the percentage outflow volume after Rainfall Event 1 was less than after Events 2 and 3. Event 1 consisted of the largest rainfall volume; the lower outflow volume percentage implies either that much of the rain water did not enter the pavement, or that the outflow reached a maximum value determined by the capacity of the drainage layers and the drainage system. The capacity of some of the outlet pipes, which also influence the time to drain, was found to be inadequate; this also likely contributed to the difference in the times to drain sections.

These observations demonstrate the importance of keeping the drainage layer and drainage system free from contamination of fines, and ensuring that the capacities of the elements of the drainage system (drainage layers, edge drains, and outlets) are matched and adequate.

3.4.1.4 Finite Element Modeling of the Test Sections

A finite element idealization was developed for the pavement to simulate the known field conditions and to aid in the prediction of the drainage performance for other conditions as well. The analyses were performed using intensity and duration of a measured rainfall event.

Cracks were also introduced into the finite element simulation. The resulting analyses showed that surface cracks were a major source of water infiltration. Predicted outflow from a cracked surface increased considerably compared to that of an uncracked surface.

3.4.2 Ontario, Canada, Study

The Ontario study was concerned with the performance of open-graded drainage layers as part of three pavement structures (Table 3.1). Little information is presented in the report on rainfall conditions, prior saturation of the pavement layers, or measurements of outflow. However, it was reported that some sections were saturated for up to 12 hours after rain had stopped. The authors suggested, however, that if the drainage system were more efficient, the saturation period could possibly have been shorter.

This study also highlighted the necessity for the drainage system to have matched and adequate components and emphasized the importance of an adequate outlet capacity to take advantage of the high permeability of the drainage layer.

This section discusses some results of this study, especially the need for a filter to retard or prevent the migration of fines and observations of stripping of the asphalt treated drainage layers.

3.4.2.1 Filter

In the test sections investigated, it was found that where water infiltrates pavement underneath cracked surfaces, fines from the base migrated into the drainage layers. This resulted in a reduction in the permeability of the layer and therefore decreased the drainage effectiveness. In one of the sections the drainage layer was placed on a granular base that had been primed and sanded (Section 1), and this section showed little contamination from the base layers.

3.4.2.2 Stripping

Stripping of the asphalt treated drainage layers was also reported, but was not considered a problem given that the asphalt is added to provide stability during construction and not for structural capacity.

Stripping occurred at locations near the cracks where the surface water infiltrated the pavement, the same areas that were contaminated by fines from the underlying layers. Generally, the amount of stripping and fines contamination were greatest at the bottom of the drainage layer. The bottom of the layer is most likely to experience the greatest amount of water flowing through or standing in it, and therefore this observation is not surprising. No data are presented to support whether the fines contamination and stripping are primarily a result of standing water or of water flowing through the pavement. It is reported that the drainage layer functioned even when the layer was contaminated with fine materials from the underlying layers. However, fines contamination is reported to reduce drainage layer capacity.

3.5 Summary and Evaluation

Based on the studies reported herein and comparisons of drainage layer usage and performance in Indiana, Ontario (Canada), and California, a number of generalizations appear warranted.

1. A major source of water infiltration into the pavement is through cracks in the pavement surface. Accordingly, it may be justified to include drainage layers in a pavement structure even when water infiltration from the subgrade is not anticipated.

2. Material specifications for the drainage layers differ among the three agencies. These differences most certainly will influence the performance of the drainage layer.

3. The most important factors influencing the efficiency of drainage layers are the capacity of the drainage system and the use of filters to prevent fines from migrating into the drainage layer.

It should also be noted that stripping of the asphalt binder in the treated drainage materials was observed in Ontario but not considered a problem and was not observed in Indiana. Also, neither the Indiana nor the Ontario studies reported the results of laboratory tests and associated conditioning to evaluate the effectiveness of the materials used in the drainage layer.

3.5.1 Water Infiltration Through Cracks

Cracks in the surface courses of pavements are a major source of water infiltration into the pavement. If timely surface maintenance is performed, the amount of water reaching the drainage layer can be reduced, thereby diminishing problems associated with water in the pavement. Even with timely maintenance, it is likely some water will infiltrate the pavement. Accordingly, the inclusion of a drainage layer can improve the ability of the pavement to drain the water in locations where sufficient rainfall occurs to alter the strength of pavement materials.

3.5.2 Material Specifications

The material used in the California asphalt treated permeable base (ATPB) has a grading much more open than the Indiana drainage materials and slightly less open than that used in Ontario.

The asphalt content used in California ATPB is approximately the same as that used in Ontario drainage materials and is less than that used in Indiana. It is possible that a higher asphalt content in the mix than that currently used in California will prevent or diminish stripping.

3.5.3 Drainage System Capacity

All of the studies have indicated that the drainage layer alone will not guarantee improved pavement performance, and that the entire drainage system must have sufficient capacity and work efficiently for improved pavement performance. This requires that the drainage layer, the outlet drains, and the outlet pipes have matched and adequate capacity for the expected water infiltration. Regular maintenance of the drainage system is necessary to ensure that when a rainfall event occurs, the water that infiltrates the pavement can be drained out as quickly as possible and before excessive pavement damage occurs.

3.5.4 Use of Filter Layer to Retard Migration of Fines into Drainage Layer

Both the Indiana and Ontario studies reported the presence of fines in the drainage layer from the untreated underlying layers. These fines reduce the permeability of the drainage layer and subsequently increase the time to drain the infiltrated water, which can, in turn, accelerate pavement damage.

Both studies recommend the use of a filter layer to prevent the migration of fines. Caltrans currently primes the untreated base before constructing the ATPB, which is thought to act as a filter. However, no data are available to validate the effectiveness of the prime as a filter. It is therefore recommended that data be obtained to evaluate the efficiency of the prime as a filter, and, if not effective, a "designed" filter should be provided, examples of which include a primed and sanded base and a treated aggregate filter.

3.5.5 Conditioning for Laboratory Testing

The Indiana and Ontario studies provided little guidance as to a conditioning procedure that could be used for laboratory test specimens. However, the field observations of relatively long periods of saturation of the drainage layers support some of the Caltrans experience reported in Chapter 2. Accordingly, it was decided to examine the behavior of ATPB specimens subjected to saturated conditions for up to ten days. Specific details of the conditioning procedures are described in Chapter 4.

4.0 LABORATORY TESTING OF ASPHALT TREATED PERMEABLE BASE

The stiffness and permanent deformation behavior of two asphalt treated permeable base (ATPB) materials subjected to soaking and repetitive loading while saturated were evaluated as a part of the CAL/APT Goal 1 Study. The two ATPB materials investigated were: 1) the material used in the test pavements 500 RF and 502 RF at the Pavement Research Center, and 2) material from Vasco Road in Contra Costa County. Measured properties included resilient modulus (M_R) and permanent deformation under repetitive loading. The purposes of this testing were as follows:

- to measure the stiffness of the materials under various states of compressive stress,
- to measure the damage (change in stiffness) to the material caused by soaking, and loading while saturated,
- to measure vertical permanent deformation when subjected to a repetitive stress condition representative of that induced by traffic loading, and
- to measure the effects of saturation on the permanent deformation.

Results of the test program are summarized in this chapter while Appendices B and C contain the detailed stiffness data for specimens prepared from the Goal 1 ATPB material and from the Vasco Road ATPB respectively.

4.1 Materials

The materials included in the test program were field mix collected during construction of the CAL/APT Goal 1 HVS test pavement at Richmond Field Station in April 1995 and field mix collected during construction of a new section of Vasco Road in Contra Costa County in August 1995.

4.1.1 CAL/APT Goal 1 Material

The CAL/APT Goal 1 material complies with nearly all requirements set forth by Caltrans 1992 Standard Specifications, Section 29-1.02A. *(10)* The Caltrans District 4 Laboratory and the Contra Costa County Materials Laboratory both performed quality assurance tests to confirm the compliance of the ATPB materials with Caltrans specifications. Quality assurance testing results for the aggregate gradation are shown in Table 4.1. The sampled gradations and specification limits are plotted in Figure 4.1. In addition to the grading requirements, the aggregate had to conform to other quality requirements, shown in Table 4.2. It can be seen that the site sample tested by Contra Costa conformed to the specification requirements except for the 2.36 mm and 0.075 mm sieves for which the measured values were about one percent higher.

The mix design asphalt cement content was 2.5 percent by mass of aggregate. However, asphalt extractions performed by the Contra Costa County Materials Laboratory on samples collected in the field showed an asphalt content of 2.9 percent. The Caltrans District 4 Laboratory (located in Richmond) measured a 2.8 percent bitumen content from materials collected at the plant.

The Rice Maximum Specific Gravity (ASTM D2041) for the ATPB was determined by the Contra Costa County Laboratory to be 2.621. The average of two tests performed by the UCB Laboratory was also 2.621. *(10)*

Table 4.1 CAL/APT Goal 1 ATPB Gradation.

Sieve (mm)	Size (US)		Percentage Passing		
		Specification Limits	Mix Design (Caltrans District 4 Lab)	Site Sample (CC County Material Lab)	Plant Sample (Caltrans Richmond Field Lab)
25.4	1 in.	100	100	100	100
19	3/4 in.	90-100	99	91	93
12.5	½ in.	35-65	56	51	54
9.5	3/8 in.	20-45	26	27	27
4.75	No. 4	0-10	7	8	6
2.36	No. 8	0-5	1	6	4
1.18	No.16		0	6	3
0.6	No. 30		0	5	3
0.3	No. 50		0	5	2
0.15	No. 100		0	4	1
0.075	No. 200	0-2	0	3	1

Table 4.2 Quality Tests of CAL/APT Goal 1 ATPB.

	Specification	Caltrans District Mix Design	Plant Data by Caltrans District 4 Laboratory
Percentage of Crushed Particles (Calif. Test Method 205)	Min 90%	100%	N/A
Los Angeles Rattler Loss at 500 Rev. (Calif. Test Method 211)	Max 45%	19%	N/A
Cleanness Value (Calif. Test Method 227)	Min 57	84	63
Film Stripping (Calif. Test Method 302)	Max 25%	No Stripping	N/A

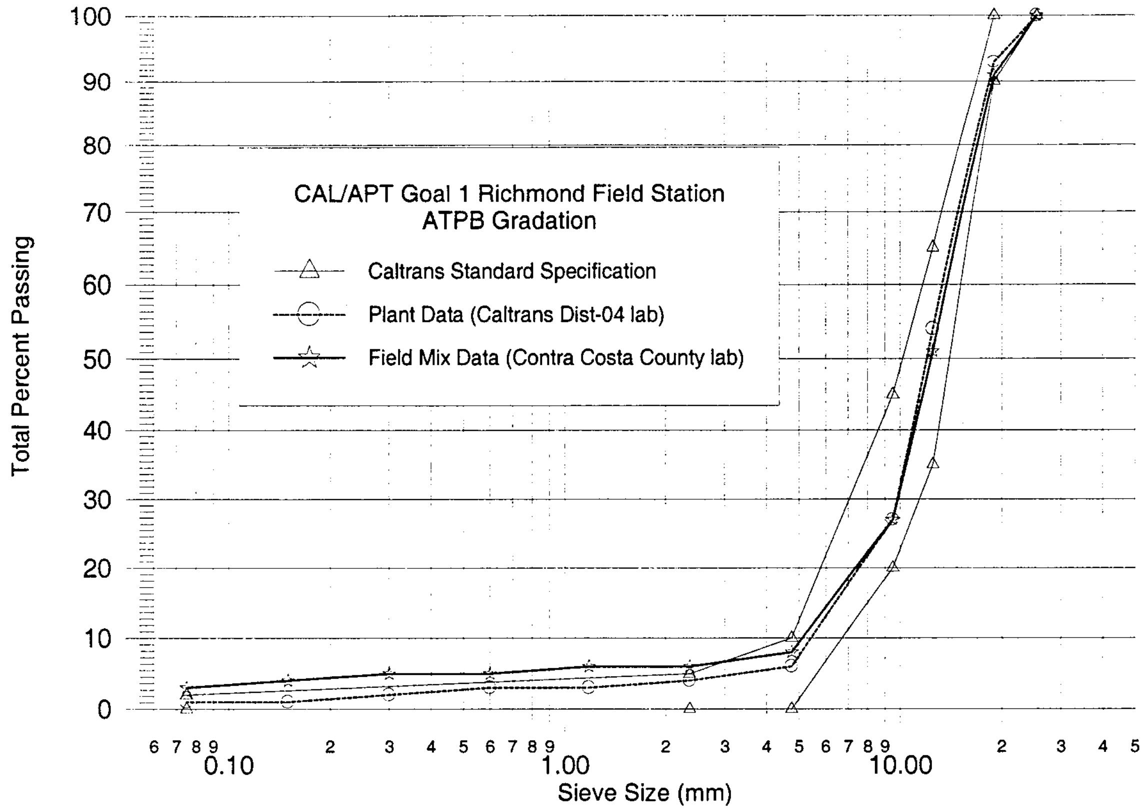

Figure 4.1. CAL/APT Goal 1 ATPB gradation test results and specification limits.

4.1.2 Vasco Road Material

Samples of ATPB were obtained from a new section of Vasco Road in Contra Costa County. The Vasco Road ATPB was specified to meet the Caltrans Standard Specifications (1992) Section 29-1.02A. *(4)* Testing for quality assurance was performed by the County Materials Laboratory.

Aggregate gradations were determined from samples collected on three separate dates: 5 July, 11 August, and 29 August 1995. Samples taken on the first two dates were obtained from the construction site while samples taken on the last date were obtained from the hot mix plant. Figure 4.2 shows the aggregate gradation for the sample collected on 29 August. The samples

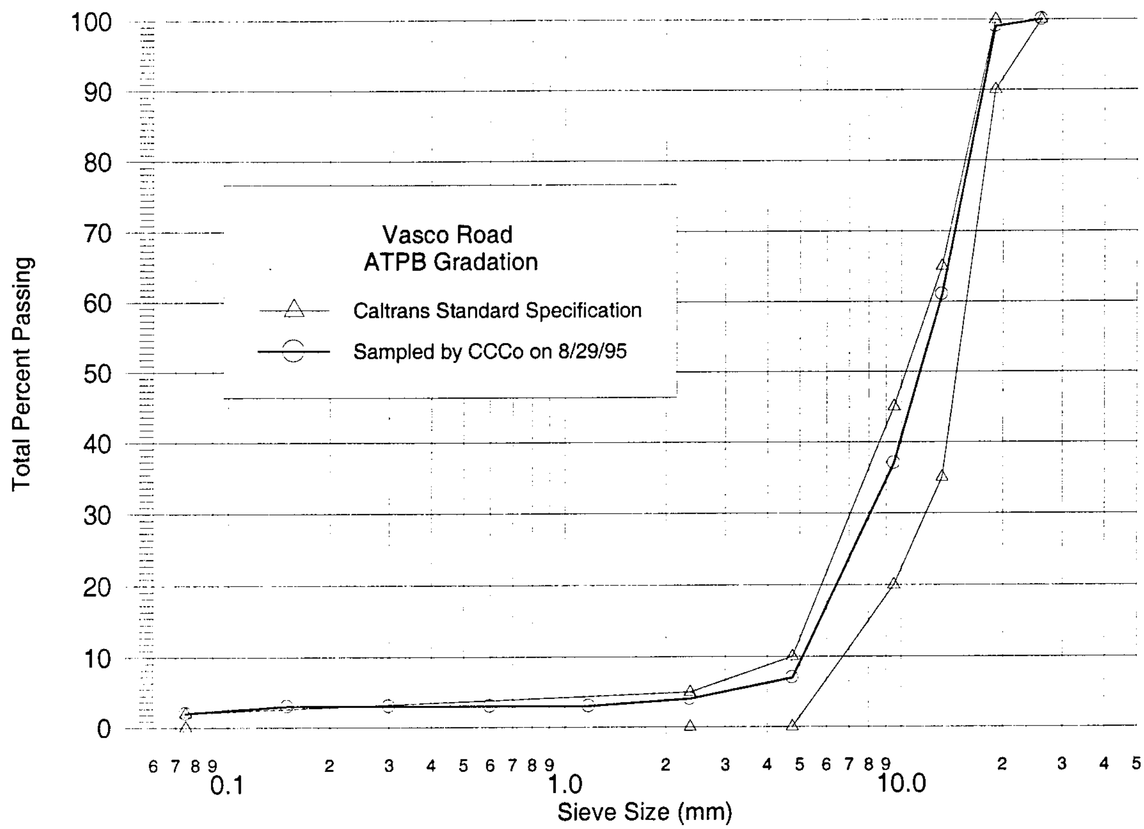

Figure 4.2. Vasco Road (Contra Costa County) ATPB Gradation Test Results and Specification Limits.

used for the preparation of test specimens were obtained after this date. In addition to the gradation requirements, other quality tests performed on the aggregate are included in Table 4.3.

The Rice Maximum Specific Gravity for samples collected on 11 August was 2.58. Changes in the bitumen content from 2.0 percent to 2.5 percent had a negligible influence on the Rice Maximum Specific Gravity. This was noted because samples sent to UCB were separated into two groups – "L" and "R" – signifying the stations where the samples were obtained. The nominal bitumen contents were 2.0 percent for "R" and 2.5 percent for "L."

Table 4.3 Test results, ATPB mix, Vasco Road, Contra Costa County.

Test	Requirement	Sampled and Tested by Contra Costa County Materials Laboratory		
		5 July	11 August	29 August
Cleanness Value	min 57	-	-	41
Bitumen Content	N/A	2.1 %	2.2 %	-
Rice Maximum Specific Gravity	N/A	-	2.58	-

4.2 Specimen Preparation and Testing

4.2.1 Specimen Preparation

The procedures for specimen preparation and set-up essentially followed those in AASHTO Designation: T274-82 (1986). *(13)* The differences were the following:

1. For specimen compaction, the plastic membrane was placed inside the split mold. The split mold with membrane inside, the rod, the cover plate, and the ATPB material (distributed in six cans) were heated in an oven at 80°C (175°F) for 1.5 to 2 hours before compaction.

2. The mixture was placed into the split mold in six lifts. Each lift was rodded followed by a vibrating load applied through a cover plate with an air hammer operating at 759 kPa (110 psi) for a period of about 60 seconds. The aggregate was found to degrade if longer compaction times were used. The vibrating frequency of the compaction hammer was about 12.5 Hz with a compaction pressure of about 5.1 kPa. Appendix

A describes an evaluation of the loading applied during specimen compaction using the vibratory hammer.

3. Following compaction, the specimen was allowed to cool overnight. The specimen was then removed from the split mold, and the air void content was measured, using Parafilm. *(14)*

4. To provide a uniform surface for load application during testing, the gaps between ATPB aggregates on the top of each specimen were filled and leveled with small gravel. The gravel material was screened so that it passed the 9.3-mm sieve and was retained on the 4.75-mm sieve.

Four specimens were prepared with the ATPB used in the Goal 1 pavement test program. These specimens have been designated as G1-1, G1-2, G1-3, and G1-4.

Similarly, four specimens were prepared from the Vasco Road material and their designations are CC5R, CC2R, CC1L, and CC2L. The "L" and "R" letters refer to the sampling locations as noted in Section 4.1.2, with the L and R specimens having asphalt contents of 2.5 percent and 2.0 percent respectively.

4.2.2 Conditioning and Test Procedures

The testing procedure for each specimen consisted of a resilient modulus test in the as-compacted state and after periods of soaking, or permanent deformation and resilient modulus tests in the as-compacted or saturated state. Each specimen was subjected to one of the following conditioning and testing sequences:

(1) Sequence 1:

$$M_R \text{ test} \xrightarrow{\text{3 days soak}} M_R \text{ test} \xrightarrow{\text{7 more days soak}} M_R \text{ test}$$

(2) Sequence 2:

$$M_R \text{ test} \xrightarrow{\text{Saturated repeated loading test for 3 days}} M_R \text{ test} \xrightarrow{\text{Saturated repeated loading test for 7 more days}} M_R \text{ test}$$

(3) Sequence 3:

$$M_R \text{ test} \xrightarrow{\text{Dry repeated loading test for 3 days}} M_R \text{ test} \xrightarrow{\text{Dry repeated loading test for 7 more days}} M_R \text{ test}$$

The apparatus for the resilient modulus test is shown in Figure 4.3.

4.2.2.1 Saturation Method

The saturation method was based on the saturation procedure from the AASHTO Designation: T274-82 (1986). *(13)* Since the material tested was ATPB, 100 percent saturation was easily obtained and was facilitated by porous stones placed on the top and bottom of each specimen. During the permanent deformation test with saturation (Sequence 2), a static hydraulic head was maintained with the water surface 10 cm to 15 cm above the top of the specimen. During this test, the specimen was maintained open to the atmosphere through the top cap valve and the hydraulic head cylinder throughout the permanent deformation test.

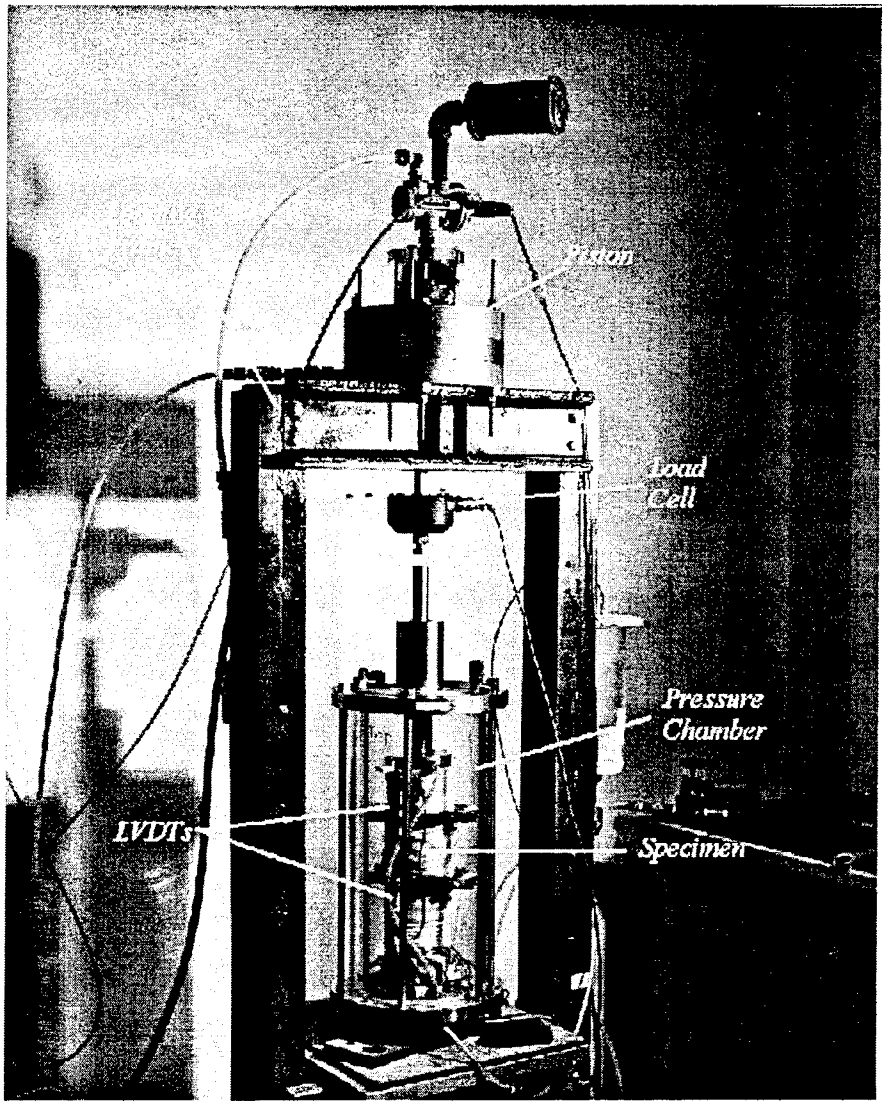

Figure 4.3 Triaxial test apparatus.

4.2.2.2 Resilient Modulus Test Method

The purpose of a resilient modulus test is to measure the stiffness characteristics of the material under different stress states, which in turn can be used for pavement analyses.

The resilient modulus of unbound granular bases, which is stress dependent *(15, 16)*, can be represented by an expression of the form: *(16)*

$$M_R = k\theta^n \tag{3}$$

where θ = bulk stress (in triaxial test $\theta = 3\sigma_c + \sigma_d$)

k, n = experimentally determined coefficients

σ_c = confining pressure in triaxial compression test

σ_d = deviator stress in compression triaxial test (applied axially)

ε_r = recoverable axial strain

M_R = resilient modulus = $\dfrac{\sigma_d}{\varepsilon_r}$

Sample conditioning was accomplished by applying various combinations of confining pressures and deviator stresses as summarized in Table 4.4.

Table 4.4 Loading Sequence Used for Pre-conditioning.

Confining Pressure		Deviator Stress	
psi	kPa	psi	kPa
10	69	12	83
10	69	30	207
15	103	30	207
15	103	45	310
20	138	45	310
20	138	60	414
30	207	60	414

The loading cycle for the conditioning test consisted of a 0.1-second impulse load and a 2.9-second rest period. For the test sequence in which the resilient modulus tests followed permanent deformation loading, the specimen was not subjected to preconditioning because it was tested continuously and never removed from the triaxial cell.

The resilient modulus tests were conducted using the sequence of confining pressures and deviator stresses summarized in Table 4.5. Two hundred load repetitions were applied at each stress state and all tests were performed at 20±2°C.

4.2.2.3 Permanent Deformation Test Method

The purpose of the ATPB in the pavement structure is to provide a permeable layer for removal of water from the asphalt concrete and base layers. If the flow of water out of the ATPB layer is prevented, for example if blockage of an edge drain occurred, the ATPB would be subjected to traffic in a saturated condition. To evaluate the stiffness and permanent deformation properties of ATPB under such conditions, repetitive loading tests were performed on saturated specimens. For comparison, similar specimens were tested as-compacted (dry) as well. The test method was also expected to determine whether stripping of the ATPB, similar to that observed by Caltrans and reported in Chapter 2, could be duplicated in the laboratory in the saturated tests.

Sequence 2 specimens were subjected to ten days of loading while fully saturated. A resilient modulus test was performed on the third and tenth days. Sequence 3 specimens were subjected to the same loading pattern, but were never exposed to water to provide a control for evaluation of the performance while saturated. For both the resilient modulus and permanent deformation tests, loading was stopped whenever failure occurred, with failure defined as the complete break-up of the specimen.

Table 4.5 Loading Sequences Used in Resilient Modulus Test.

Confining Pressure		Deviator Stress	
psi	kPa	psi	kPa
10	69	12	83
10	69	20	138
10	69	30	207
15	103	16	110
15	103	30	207
15	103	45	310
20	138	20	138
20	138	40	276
20	138	60	414
30	207	30	207
30	207	60	414
20	138	40	276
10	69	20	138
5	34	12	83
5	34	20	138
5	34	12	83
10	69	20	138
20	138	40	276
30	207	60	414
30	207	30	207
20	138	60	414
20	138	40	276
20	138	20	138
15	103	45	310
15	103	30	207
15	103	16	110
10	69	30	207
10	69	20	138
10	69	12	83

For the permanent deformation tests, the loading cycle consisted of a 0.1-second impulse load and a 2.9-second rest period. Each specimen was subjected to zero confining pressure and 138-kPa (20-psi) deviator stress. This deviator stress state was selected based on the stress state determined for the pavement section shown in Table 4.6, represented as a layered elastic system. This structure is the nearly identical to the TI = 9 drained pavement used for CAL/APT Heavy Vehicle Simulator (HVS) Goal 1 testing at the Pavement Research Center (Sections 500RF and 502CT). *(10, 17)*

The simulated load was 40 kN on dual wheels, with a center-to-center spacing of 366 mm (14.4 in.) and a uniform contact pressure of 690 kPa (100 psi). A tensile stress (σ_3) was computed under the center of the loading between the loads (Table 4.7). Tensile stresses cannot be imposed in the triaxial compression test used for this study, so the specimens were tested unconfined. The deviator stress of 138 kPa (20 psi) corresponds to that at mid depth for the ATPB layer (Table 4.7).

Table 4.6 Pavement Structural Section Model for Selection of Permanent Deformation Test Stress State.

Pavement Layer	**Modulus MPa (ksi)**	**Thickness cm (in.)**	**Poisson ratio**
Asphalt Concrete	9315 (1350)	13.7 (5.4)	0.35
ATPB	1035 (150)	7.6 (3.0)	0.40
Aggregate base/ Asphalt Subbase	241.5 (35)	39.8 (15.7)	0.35
Subgrade	69 (10)	infinite	0.35

Table 4.7 Stress Analysis for Drained Pavement Structure Model Subjected to 40-kN Load.

Location	Depth mm (in)	σ_1 (psi)	σ_3 (psi)	σ_1-σ_3 (psi)
under center line between tires				
	mm 5.9	9.5	-9.1	18.6
	mm 6.9	12.7	-8.3	20.9
	mm 7.9	16.4	-7.5	23.9
under center line of one tire				
	mm 5.9	9.4	-13.1	22.5
	mm 6.9	12.4	-10.0	22.5
	mm 7.9	16.3	-8.0	24.3
under inside edge of tire				
	mm 5.9	9.7	-10.8	20.4
	mm 6.9	12.9	-9.0	21.9
	mm 7.9	16.8	-7.7	24.5

4.2.3 Alterations to Procedures for Vasco Road Tests

The following alterations to the procedures described in Section 4.2 and its subsections were included in the test plan for the Vasco Road ATPB materials. The specimen preparation procedure was the same as the procedure used for the CAL/APT Goal 1 materials except that the compaction time of each lift was reduced to 45 seconds.

The Vasco Road specimens were tested following loading Sequence 1. However, the loading period was reduced to 50 repetitions from the 200 repetitions used for the CAL/APT Goal 1 specimens, and the conditioning test was repeated before each resilient modulus test. The rationale for the change in the test procedure was as follows: given that the specimens in loading Sequence 1 needed to be moved back and forth between triaxial cell and the water bucket used for soaking, conditioning was needed to re-seat the specimen in the machine before performing the resilient modulus test. The reduction of the loading repetitions would reduce the damage from M_R testing.

4.2.4 Testing Results And Discussion

4.2.4.1 CAL/APT Goal 1 Material

The air-void contents of the four ATPB Goal 1 specimens were similar, as shown in Table 4.8.

Table 4.8 Air-Void Contents of CAL/APT Goal 1 Specimens.

Specimen	Air-Void Content (%)
G1-1	35.7
G1-2	34.5
G1-3	34.2
G1-4	35.0
mean	34.85
S.D.	0.66

The as-compacted resilient modulus test results for the specimens are shown in Figures 4.4 through 4.7. It can be seen that there is a great deal of variance in the resilient moduli of the as-compacted tests for these four specimens; average values range from about 0.8 Gpa (1.2×10^8 psi) to 2.0 Gpa (2.9×10^5 psi) (Appendix B contains the test data used to plot these figures). It is thought that the most likely cause of the high variance is the ratio of the size of specimen to the maximum aggregate size. For this ATPB material, the aggregate size is fairly uniform and about 70 percent by mass of the aggregate is between 10 mm and 20 mm in its least dimension (Figure 4.1). The maximum aggregate size was about 25 mm. Thus, the ratio of the

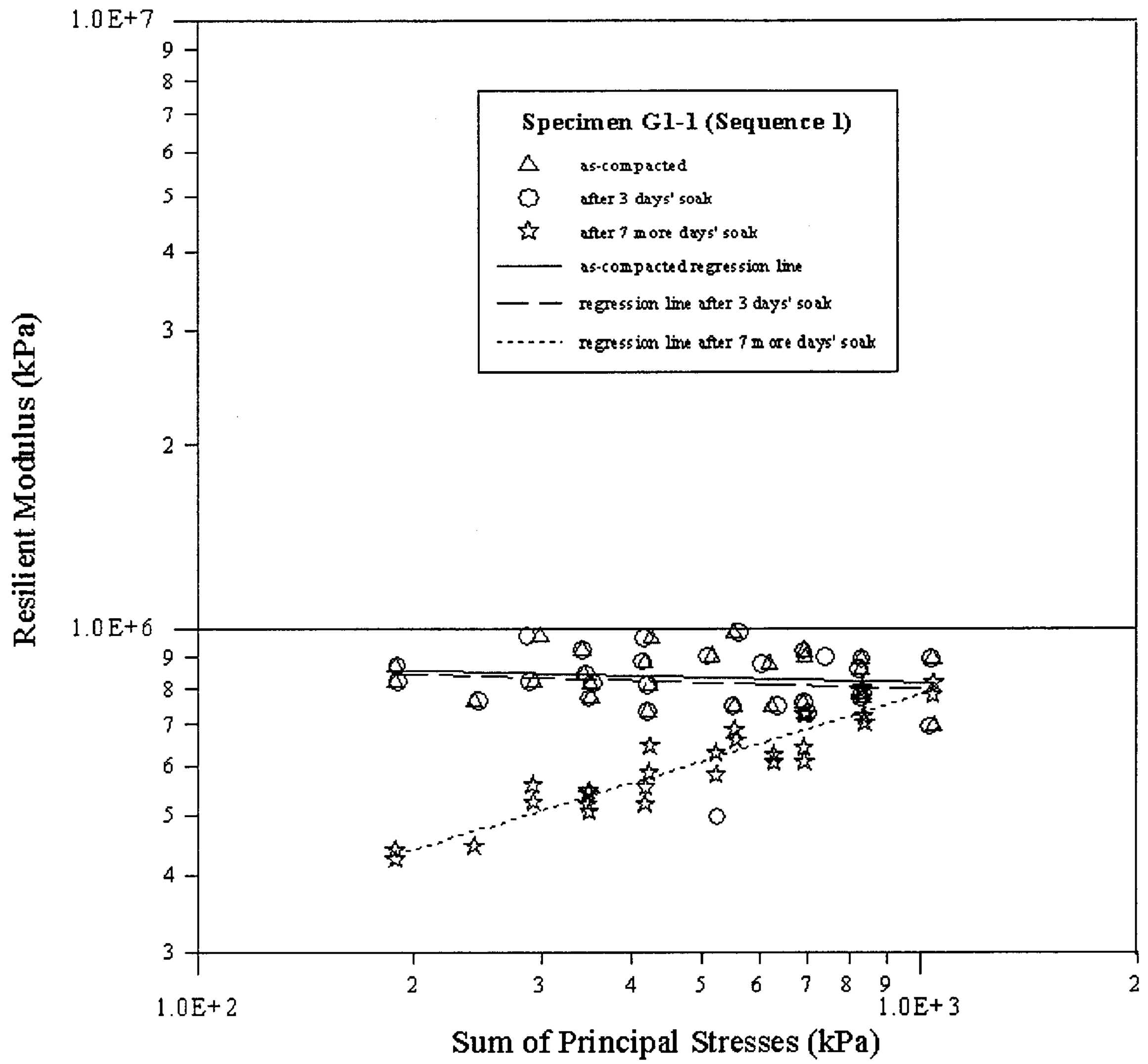

Figure 4.4. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen G1-1 with Air-Voids = 35.7 Percent (CAL/APT Goal 1).

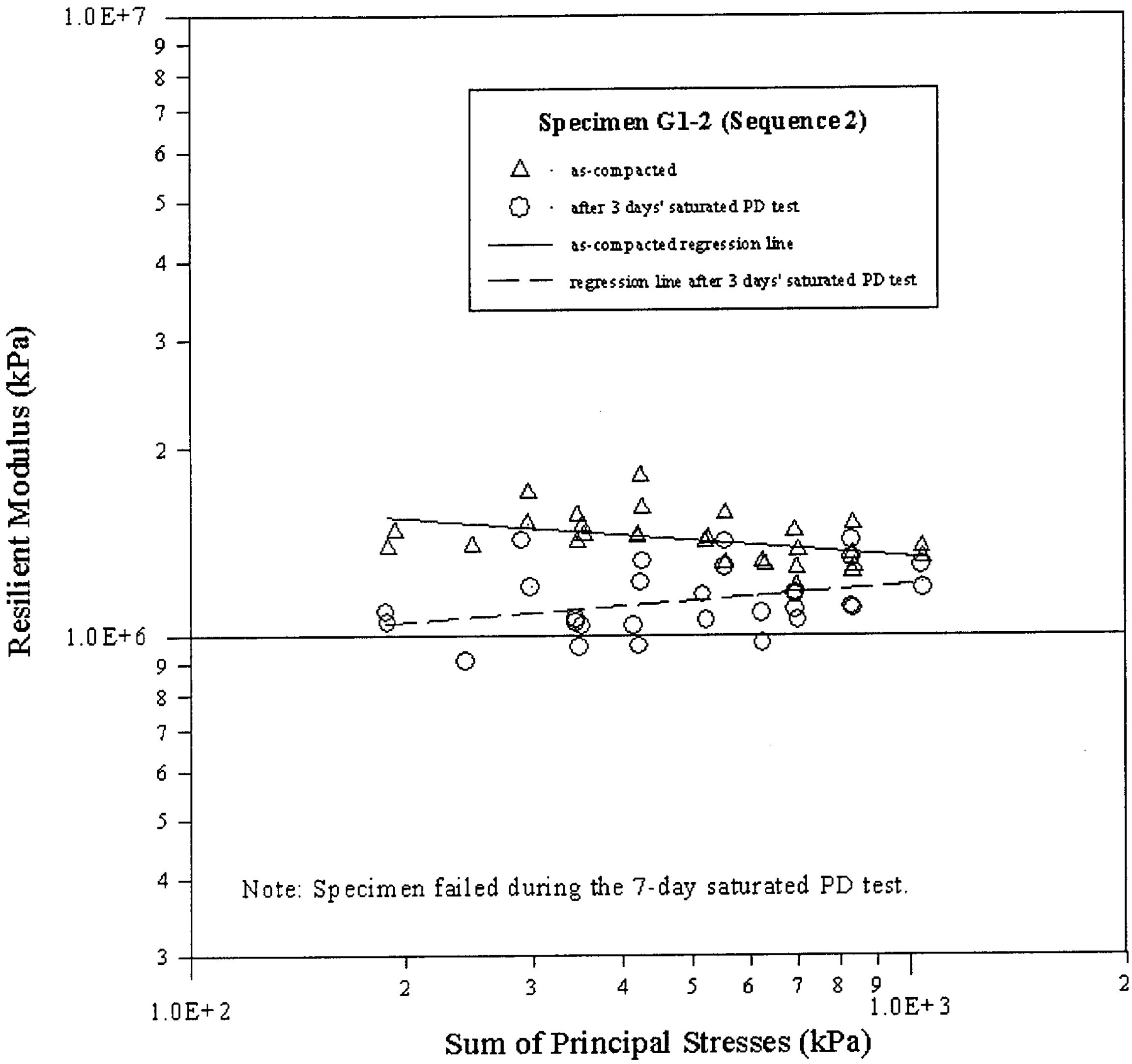

Figure 4.5. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen G1-2 with Air-Voids = 34.5 Percent (CAL/APT Goal 1).

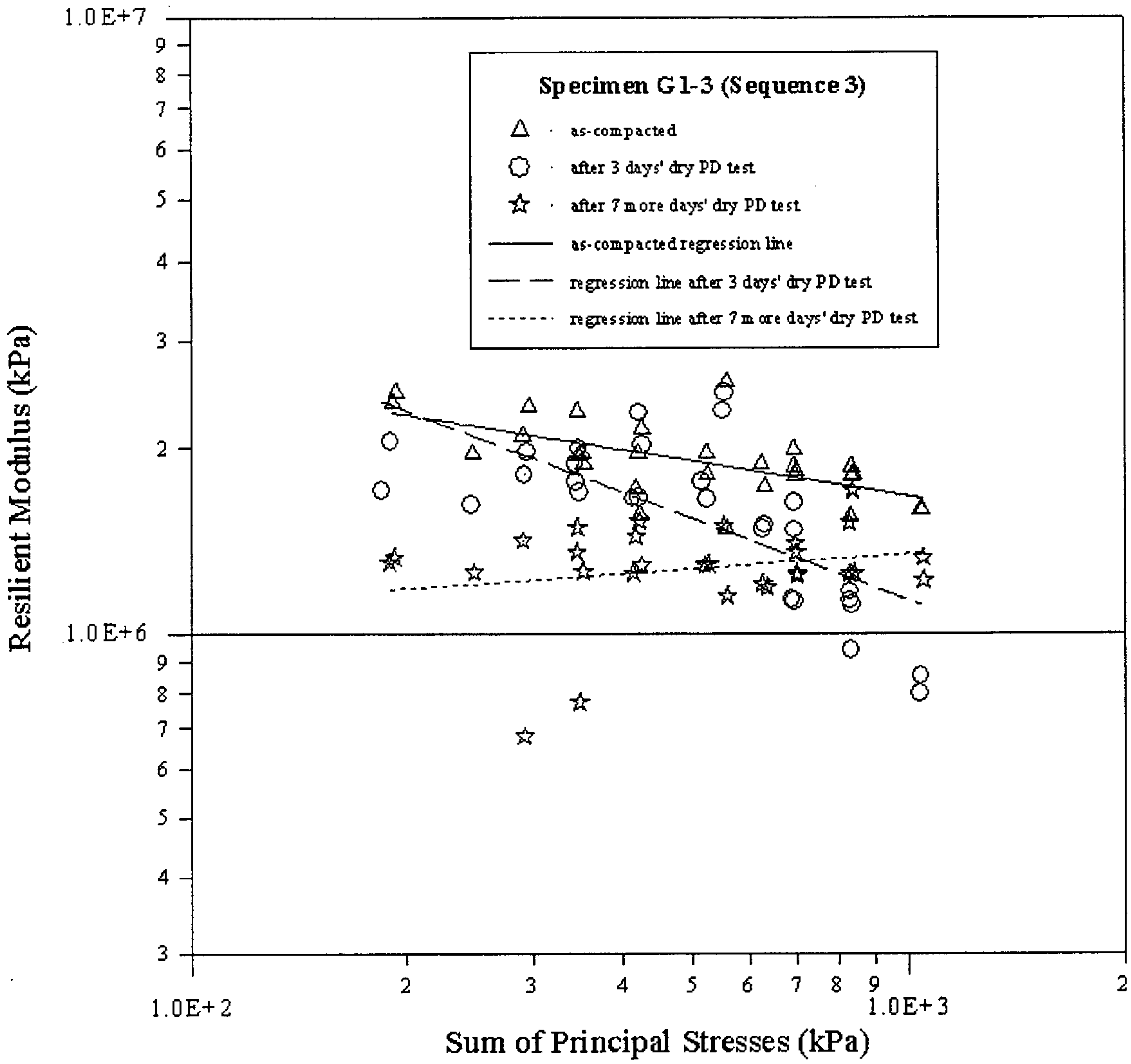

Figure 4.6. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen G1-3 with Air-Voids = 34.2 Percent (CAL/APT Goal 1).

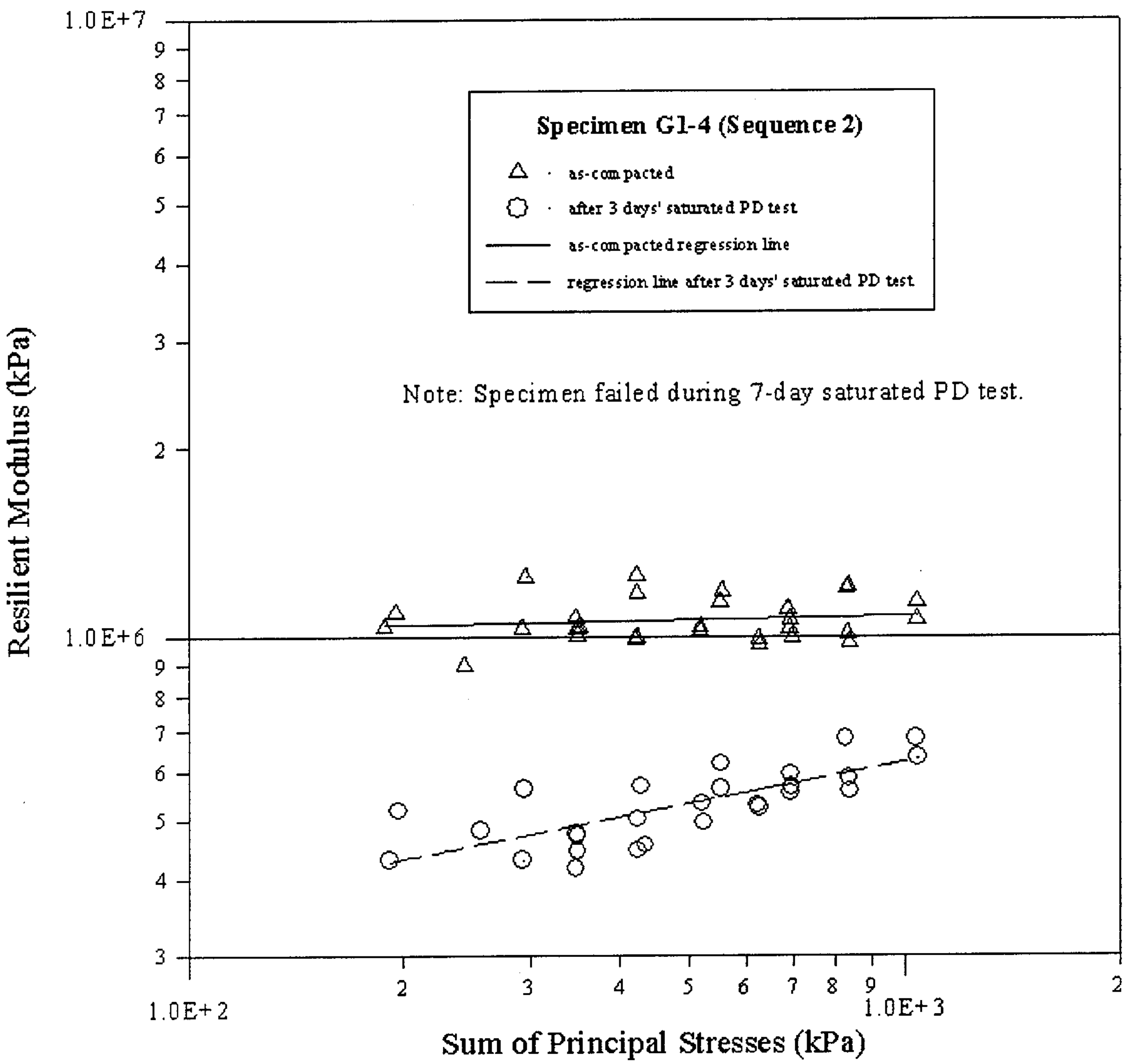

Figure 4.7. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen G1-4 with Air-Voids = 35.0 Percent (CAL/APT Goal 1).

10-cm (4-in.) diameter specimen to the 25-mm aggregate size was 4 to 1, which is generally considered a minimum for triaxial testing of granular materials.

Comparison of the as-compacted moduli and the corresponding regression lines between the sum of the principal stresses and M_R for the four ATPB Goal 1 specimens, shown in Figure 4.8, demonstrates that the higher air-void content specimens had lower as-compacted resilient moduli. Although differences in air-void content are small, these differences would be expected to have some influence on compressive stiffness.

The slopes of the regression lines between M_R and the sum of the principal stresses (SPS) for the as-compacted specimens are close to zero, indicating that the moduli in the as-compacted state are sensibly independent of stress state. Table 4.9 shows the regression results for the four specimens and their corresponding M_R values at three levels of the sum of principal stresses: 200 kPa, 500 kPa, and 1000 kPa.

Table 4.9 Summary of Resilient Modulus Test Results (kPa) for ATPB Goal 1 Specimens.

Specimen	Applied Testing Sequence	MR Testing Stages	MR0				
			MR=a*SPS^b		MR		
			a	b	SPS=200	SPS=500	SPS=1000
G1-1	1	as compacted	9.9057E+6	0.0281	853,766	832,103	816,808
		3 days soak	1.0092E+6	-0.0339	843,102	817,286	798,284
		10 days soak	6.6038E+4	0.3575	438,950	609,086	780,363
G1-2	2	as compacted	2.4542E+6	-0.0879	1,540,488	1,421,282	1,337,273
		3 days wet loading	6.6383E+5	0.0869	1,051,850	1,139,002	1,209,696
		10 days wet loading	*specimen failed before 10 days*				
G1-3	3	as compacted	6.1428E+6	-0.1884	2,263,551	1,904,619	1,671,423
		3 days dry loading	2.3506E+7	-0.4388	2,298,451	1,537,486	1,134,261
		10 days wet loading	7.7000E+5	0.0812	1,183,837	1,275,255	1,349,071
G1-4	2	as compacted	9.3633E+5	0.0207	1,044,893	1,064,907	1,080,301
		3 days wet loading	1.2790E+5	0.2294	431,278	532,168	623,890
		10 days wet loading	*specimen failed before 10 days*				

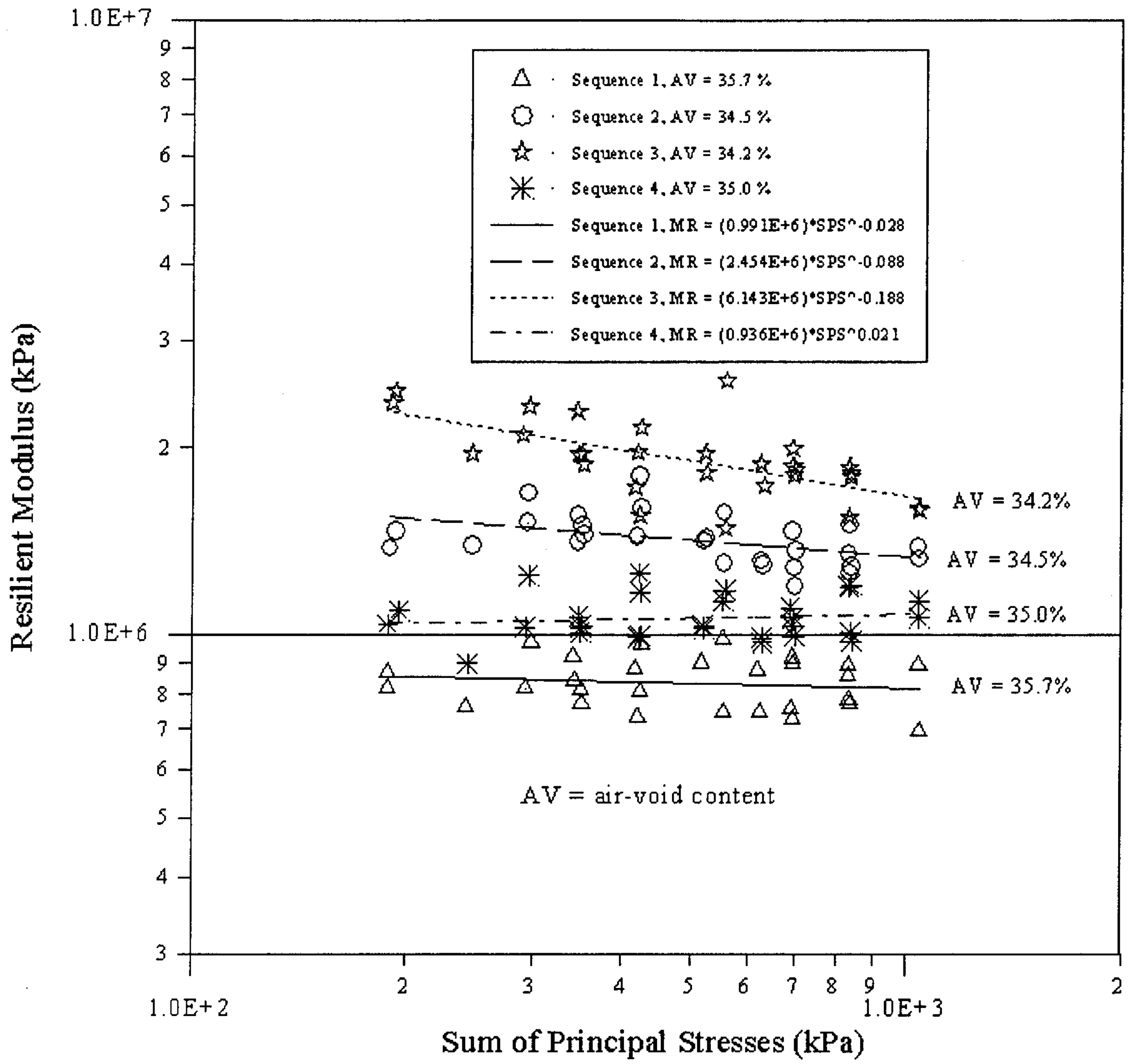

Figure 4.8. As-Compacted Resilient Moduli and the Corresponding Regression Lines for Specimens G1-1, G1-2, G1-3, and G1-4 (CAL/APT Goal 1).

For the specimens subjected to soaking (Sequence 1), shown in Figure 4.4, the average M_R at an SPS of 800 MPa shows little difference between the specimen as compacted and after 3 days of soaking. However, after 10 days of soaking, the resilient moduli were reduced and became sensitive to the sum of the principal stresses (SPS). The slope of the regression line after 10 days soaking indicates behavior similar to that of an unbound gravel, with M_R increasing with increased confinement. Although no replicate tests were performed for this sequence and this material, the results indicate that soaking without loading results in a change in the resilient moduli of the ATPB.

The resilient modulus test results for the two replicate specimens subjected to Sequence 2 are shown in Figures 4.5 and 4.7. The average as-compacted resilient modulus was 143 MPa and 106 MPa for the two replicates. Both specimens failed during the saturated permanent deformation test. Specimen G1-2 failed at 201,600 repetitions and Specimen G1-4 failed at 186,400 repetitions. Both specimens had reduced M_R values after 3 days of repetitive loading while saturated and a change to a relation between M_R and SPS similar to that of unbound gravel materials. This change occurred after 3 days of loading while saturated compared to a similar change for Specimen G1-1 after 10 days of soaking without loading.

Figure 4.6 shows the resilient modulus results for Specimen G1-3, which was subjected to Sequence 3 (the dry permanent deformation test). Compared to the specimens tested using Sequences 1 and 2, the specimens tested using Sequence 3 had greater variance in each of the three resilient modulus tests, especially after 10 days of permanent deformation testing. It is apparent from the results that the dry permanent deformation test damaged the material; however, the damage was less than that which resulted from the saturated permanent deformation tests.

A phenomenon common to all four CAL/APT Goal 1 specimens is that the slopes of the regression lines for M_R versus SPS changed from negative or zero to positive as the specimens were subjected to soaking, repetitive loading, or a combination of both. As stated earlier in this section, a positive slope indicates that the material tends to behave as an unbound aggregate. This slope change therefore indicates that the bond between asphalt-coated aggregate particles has been damaged to some extent and plays a diminished role in the behavior of the material.

The two specimens subjected to loading while saturated exhibited more permanent axial deformation than did the specimen subjected to loading in the as-compacted (dry) condition, as can be seen in Figure 4.9.

The axial deformations of both specimens subjected to Sequence 2 tended to increase rapidly after 100,000 load repetitions, or 3 days, when the second M_R test was performed. Inspection of the specimens after failure revealed that the center half of the specimens had a complete loss of bonding between aggregates (Figure 4.10), and that at some of the contact points between aggregates the asphalt had stripped from one or both of the aggregate surfaces (Figure 4.11). This condition appears to simulate the stripping observed in the field section to some degree, as discussed in Chapter 2.

Specimen G1-4, which had a slightly greater air-void content and a lower initial M_R, exhibited more permanent deformation versus load repetitions and failed sooner than did Specimen G1-2. This difference in performance could be due to the difference in degree of compaction, the difference between the material samples, or a combination of these two factors.

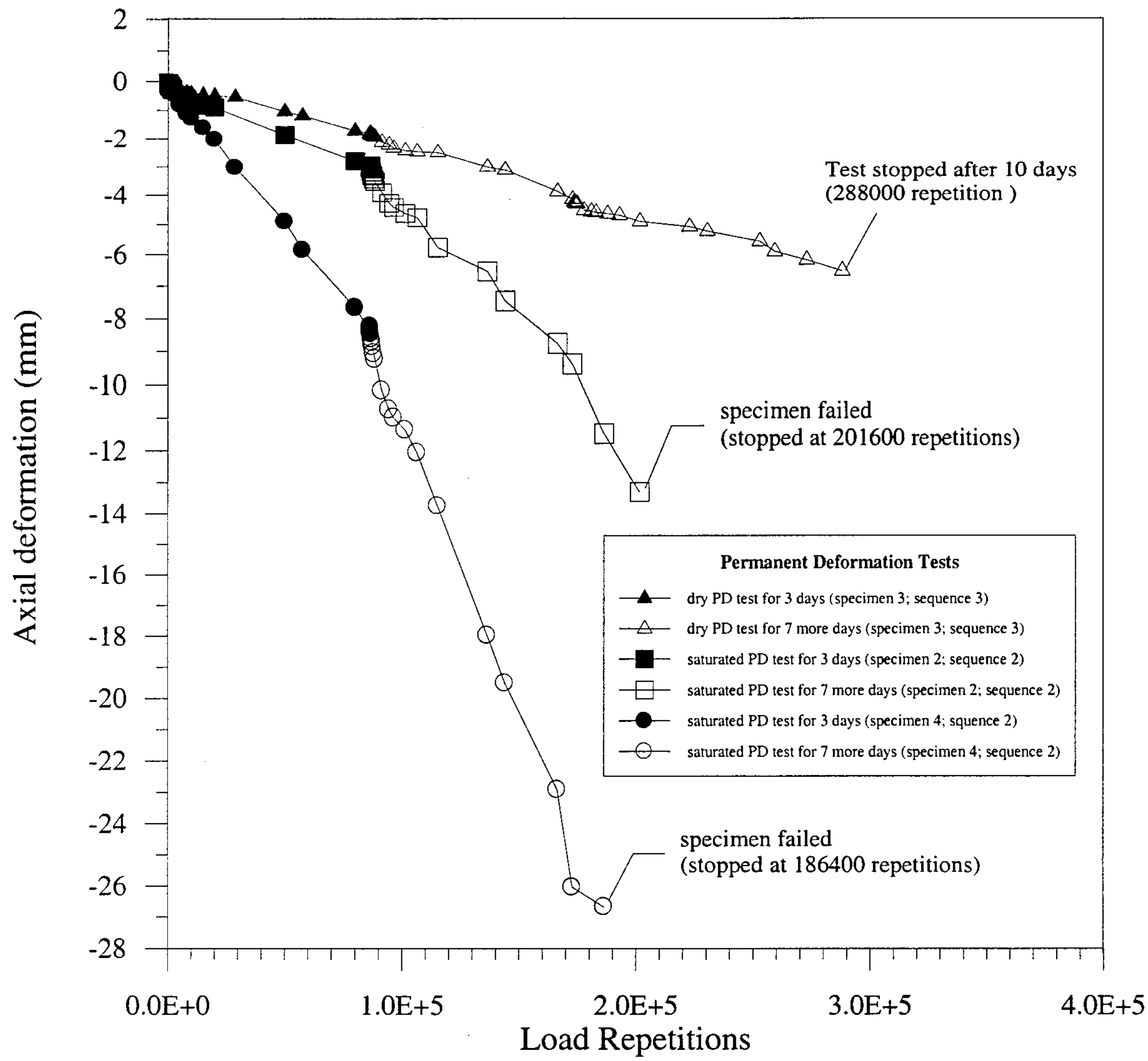

Figure 4.9. Permanent Deformation Test Results for ATPB Specimens G1-2, G1-3, and G1-4 Subjected to a Pulse Loading of 138 kPa (20 psi) and Zero Confining Pressure (Loading Time: 0.1 Seconds; Rest Period: 2.9 Seconds).

Figure 4.10. Goal 1 ATPB Specimen Failed in Saturated Permanent Deformation Test.

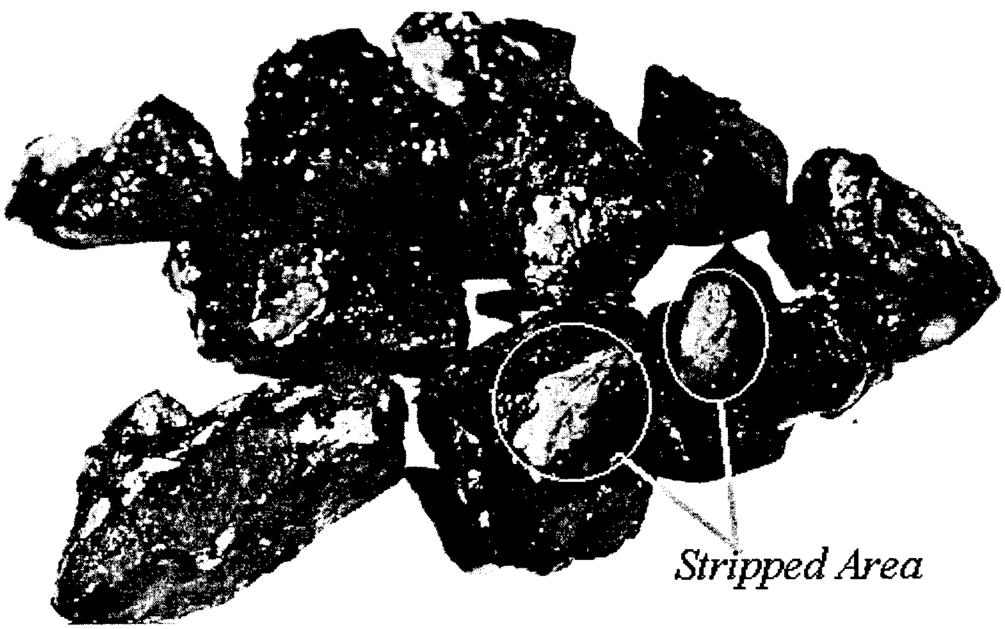

Figure 4.11. Stripping at Aggregate Interfaces in Goal 1 ATPB Specimen Failed in Saturated Permanent Deformation Test.

4.2.4.2 Vasco Road Material

The air-void contents and asphalt contents for the four Vasco Road specimens are shown in Table 4.10.

Table 4.10 Air-Void Contents and Nominal Bitumen Contents of Vasco Road Specimens.

Specimen	Rice	Nominal Asphalt Content (%)	Air-Void Content (%)
CC5R	2.58	2.0	32.1
CC1L	2.58	2.5	34.8
CC2R	2.58	2.0	32.5
CC2L	2.58	2.5	32.4

The resilient modulus test results for the four specimens are shown in Figures 4.12 through 4.15; detailed test results are included in Appendix C. Specimens CC5R and CC2R, with a nominal asphalt content of 2.0 percent, show some sensitivity to ten days of soaking. On the other hand, Specimens CC1L and CC2L, with a nominal asphalt content of 2.5 percent, do not exhibit this sensitivity. Although the sample size of this study was too small to make statistical statements and the variance of the results is large, increased asphalt content appears to reduce the damage caused by soaking. Table 4.11 shows the regressions between M_R and SPS for the four specimens and their corresponding M_R values at three values of sum of the principal stresses: 200 kPa, 500 kPa, and 1000 kPa. There does not appear to be any difference in as-compacted stiffness as a result of changes in asphalt content directly, as can be seen in Figure 4.16 and Table 4.11.

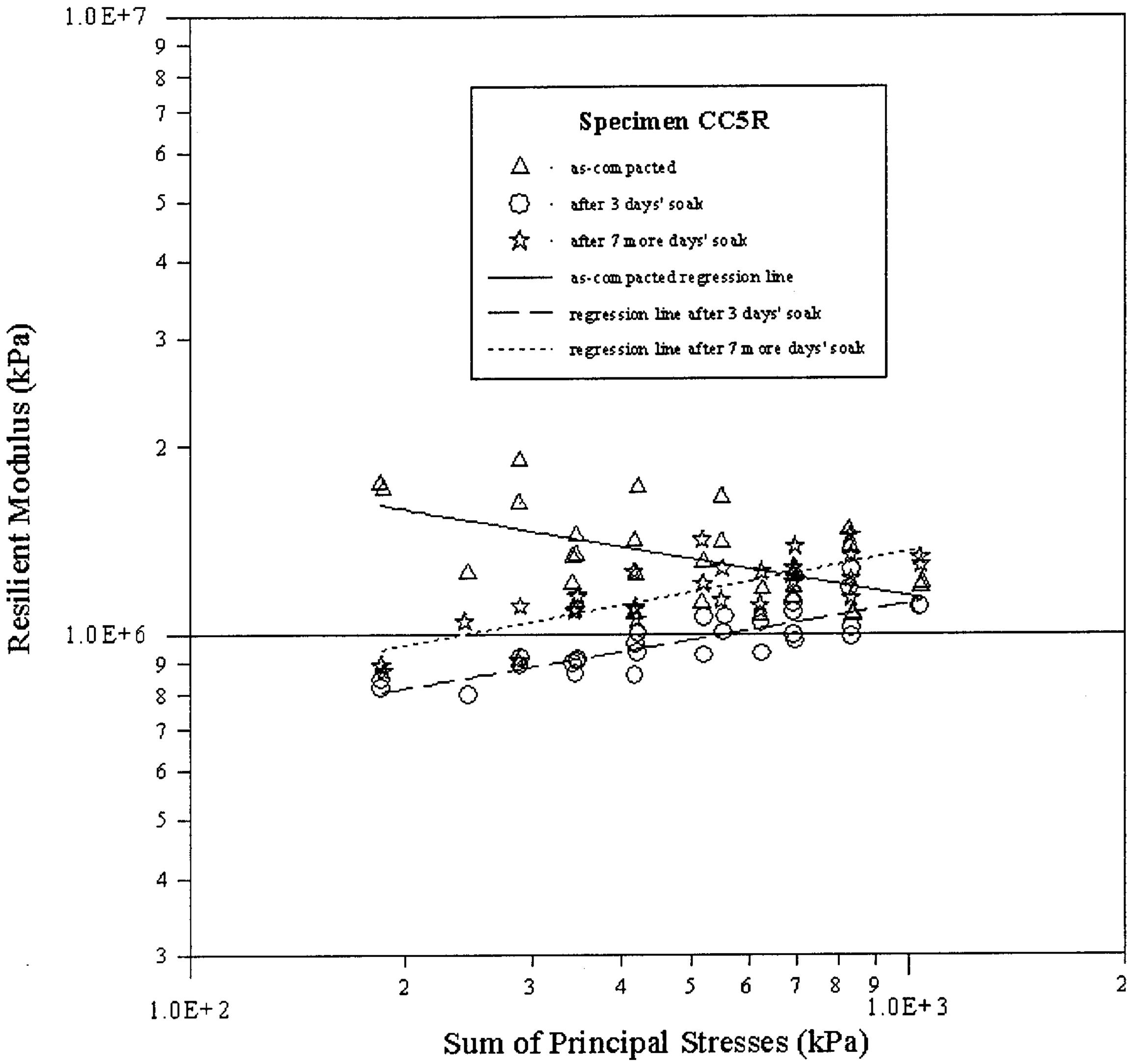

Figure 4.12. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen CC5R with Air-Voids=32.1 Percent and Nominal Asphalt Content=2.0 Percent (Vasco Road).

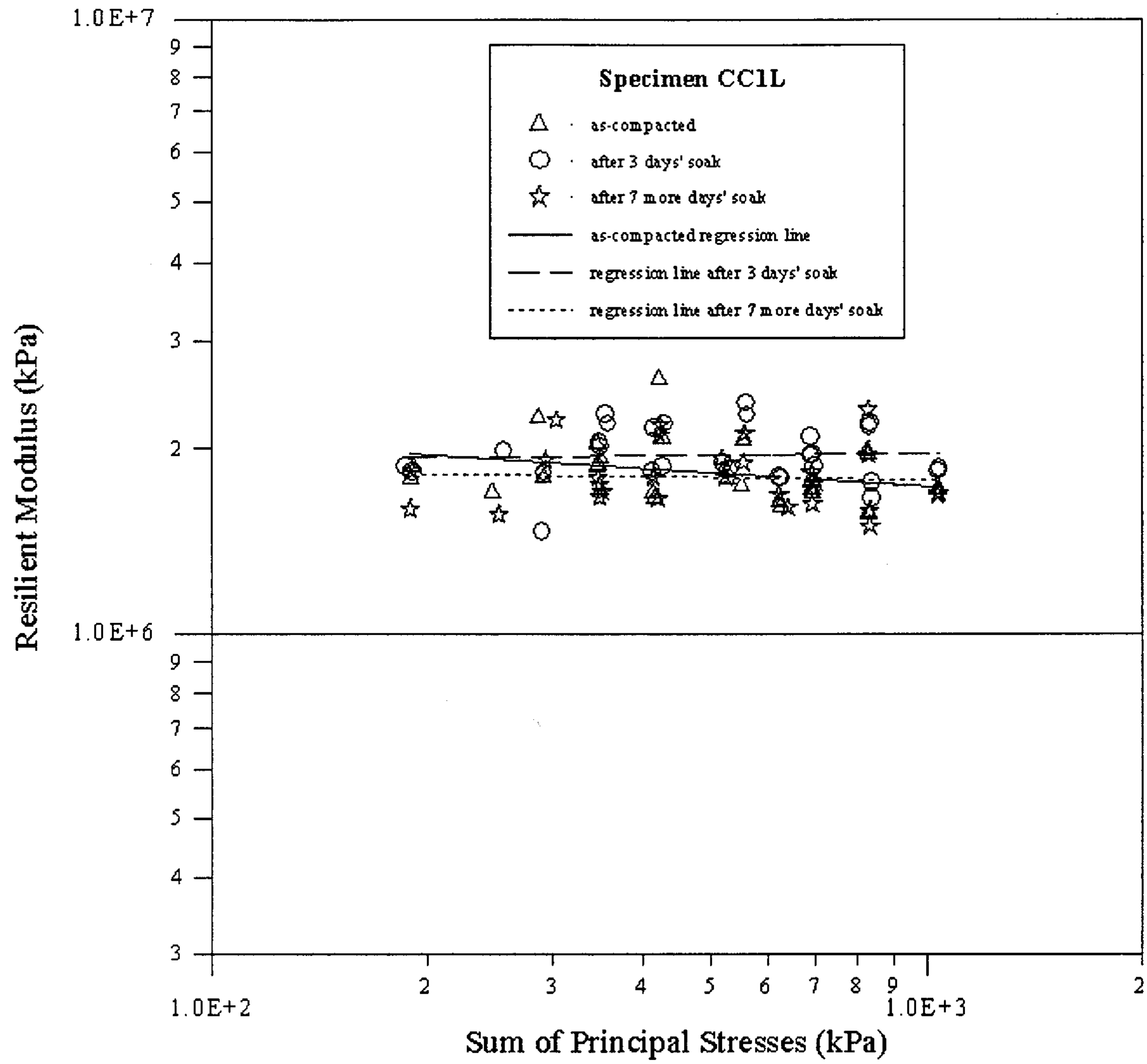

Figure 4.13. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen CC1L with Air-Voids=34.8 Percent and Nominal Asphalt Content = 2.5 Percent (Vasco Road).

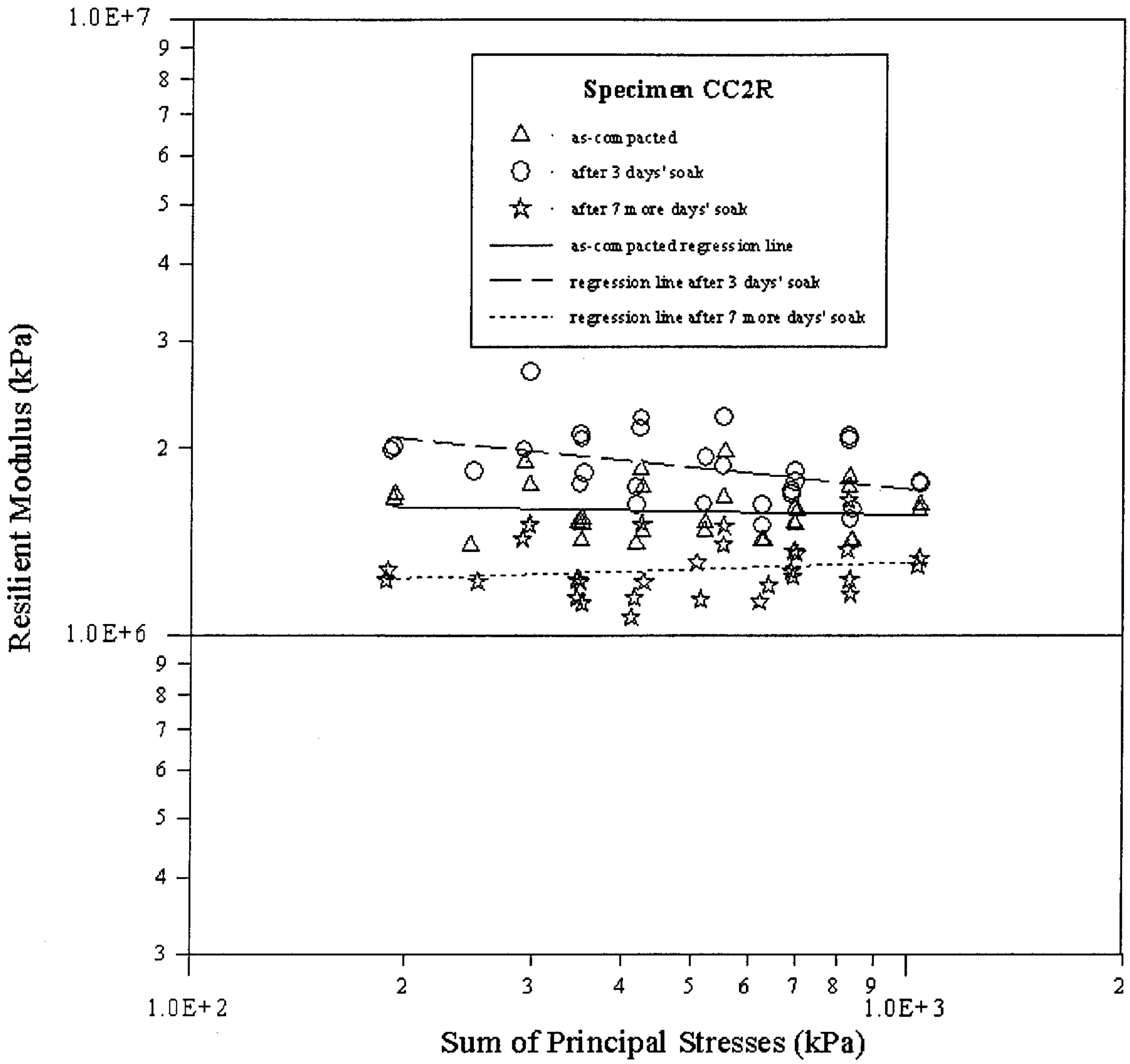

Figure 4.14. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen CC2R with Air-Voids = 32.5 Percent and Nominal Asphalt Content = 2.0 Percent (Vasco Road).

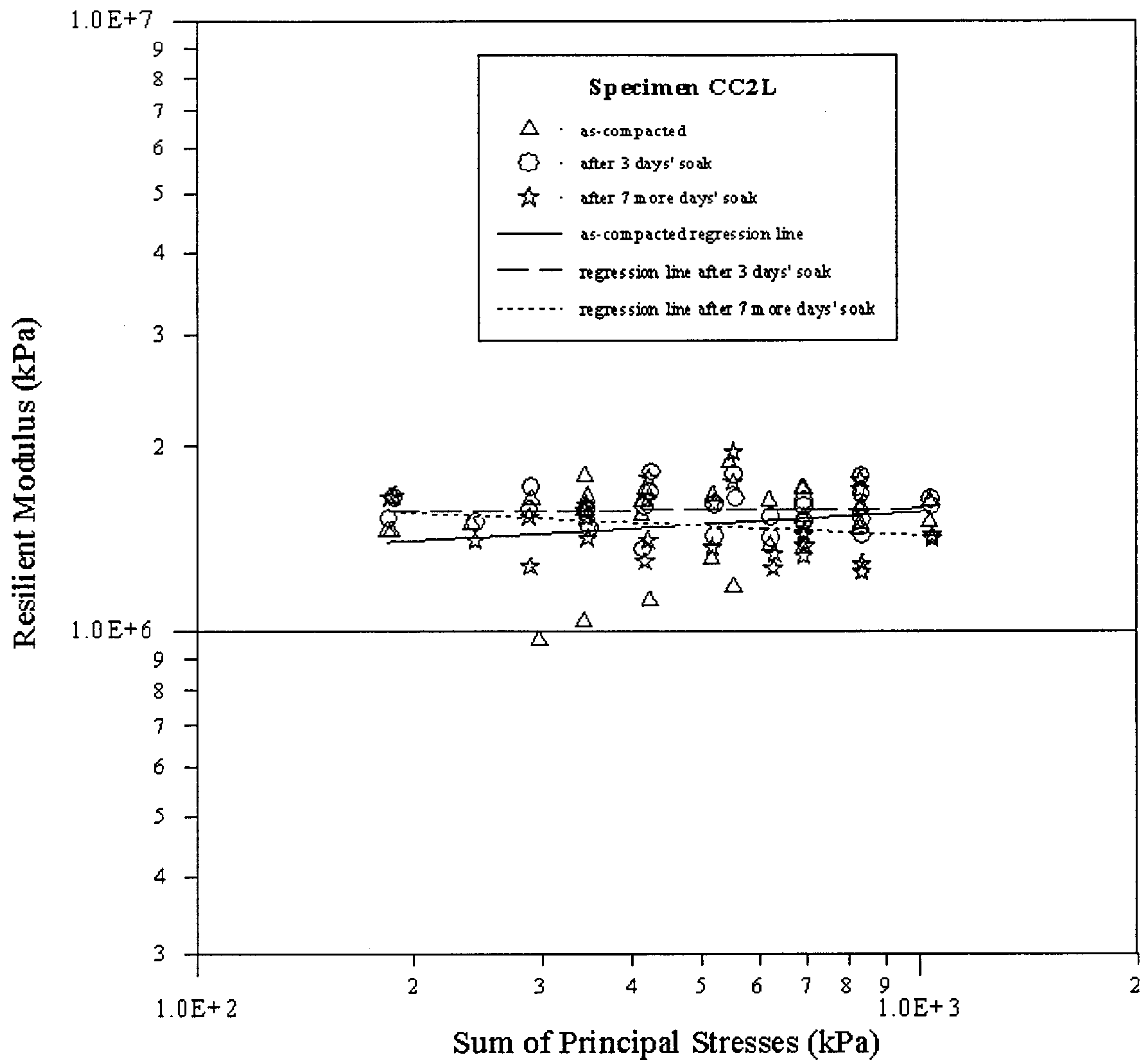

Figure 4.15. Resilient Modulus Test Results and the Corresponding Regression Lines for Specimen CC2L with Air-Voids = 32.1 Percent and Nominal Asphalt Content = 2.5 Percent (Vasco Road).

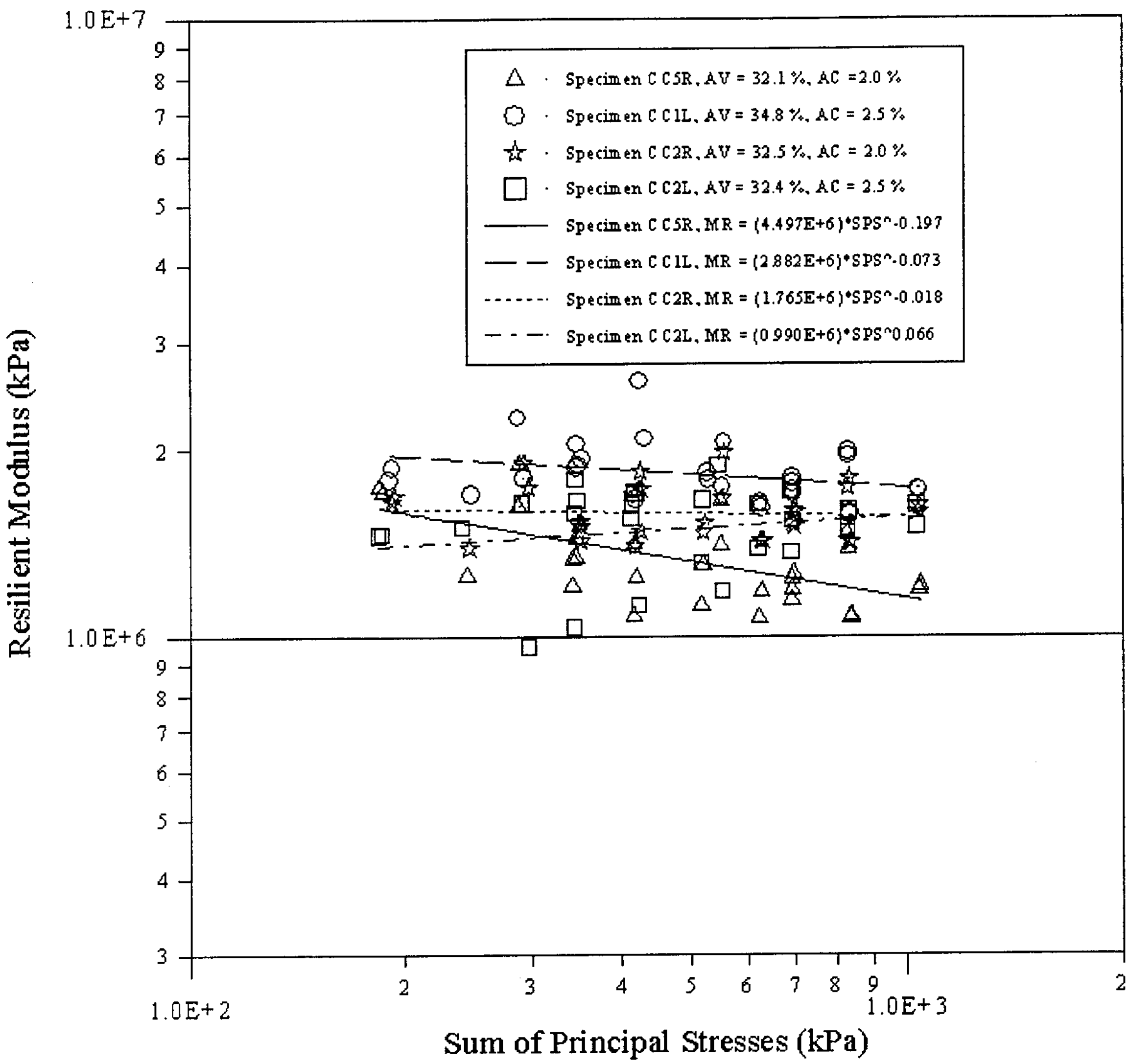

Figure 4.16. As-Compacted Resilient Moduli and the Corresponding Regression Lines for Specimens CC5R, CC1L, CC2R, and CC2L (Vasco Road).

Table 4.11 Summary of Resilient Moduli for Vasco Road Specimens.

Specimen	Testing Sequence	M_R Testing Stages	M_R0				
			M_R=a*SPS^b		M_R		
			a	b	SPS=200	SPS=500	SPS=1000
CC5R	1	as-compacted	4.4967E+6	-0.1970	1,583,520	1,322,012	1,153,286
		3 days soak	2.8736E+5	0.1973	817,451	979,449	1,123,002
		10 days soak	3.0796E+5	0.2149	961,510	1,170,753	1,358,789
CC1L	1	as-compacted	2.8818E+6	-0.0733	1,954,681	1,827,769	1,737,268
		3 days soak	1.842E+6	0.0095	1,937,527	1,954,468	1,967,381
		10 days soak	1.9577E+6	-0.0136	1,821,266	1,798,656	1,781,739
CC2R	1	as-compacted	1.7651E+6	-0.0177	1,607,003	1,581,137	1,561,847
		3 days soak	3.8391E+6	-0.1163	2,073,591	1,864,060	1,719,740
		10 days soak	1.0218E+6	0.0361	1,237,224	1,278,844	1,311,255
CC2L	1	as-compacted	9.8982E+5	0.0663	1,406,398	1,494,483	1,564,763
		3 days soak	1.505E+6	0.0079	1,569,172	1,580,546	1,589,205
		10 days soak	2.059E+6	-0.0521	1,562.373	1,489,488	1,436,621

The slope of the regression relation between M_R and SPS for the specimens with 2.0 percent asphalt content tended to become positive and more sensitive to the SPS after ten days soaking. As noted earlier, the Goal 1 ATPB material exhibited the same trend.

The as-compacted resilient moduli and corresponding regression lines for the Vasco Road materials, shown in Figure 4.16, show an increase in modulus with decreased air-void content similar to that observed for the Goal 1 ATPB material. The significance of this observation is not clear, given the small sample size and small difference between air-void contents. However, it was observed for both materials that better compaction produced greater stiffness, and in the case of the Goal 1 ATPB material, greater resistance to permanent deformation while saturated.

4.3 Evaluation

The results of the laboratory tests of the two ATPB materials designed to Caltrans specifications provide an indication of the as-compacted stiffness of the material and the potential for damage when subjected to repetitive loading in the as-compacted (dry) state, or soaking and repetitive loading while saturated. This laboratory test program can be summarized as follows:

1. The as-compacted laboratory stiffness of ATPB ranged from 800 MPa to 2,200 MPa (116 to 319 ksi) at 20°C (68°F). At this temperature, ATPB exhibits stiffnesses one-half to one-fourth that of a typical Caltrans asphalt concrete mix, and is four times stiffer than typical Caltrans base materials (Class 2 aggregate base or Class B cement treated base).

2. The ATPB materials appear to be susceptible to damage, (i.e., reduction of stiffness), when soaked in water at 20°C without loading, and when they are subjected to repetitive loading while saturated. The extent of damage is dependent on the aggregate type and the amount of asphalt. Damage observed after repetitive loading in a dry condition was less than the damage caused by soaking at 20°C without loading or by repeated loading while saturated.

3. Limited tests on the Vasco Road ATPB suggest that an increase in asphalt content results in less damage (reduction in stiffness) when soaked for a period of time. These results tend to support the effects of asphalt content reported in Chapter 3.

4. Based on test results on Goal 1 ATPB specimens subjected to repetitive loading while saturated, stripping appears to contribute to the loss of stiffness under those

conditions. Visually observable stripping and complete loss of cohesion between aggregate particles occurred in the materials after six to seven days of repetitive loading (186,000 to 200,000 repetitions) while saturated. Such results support field observations reported in Chapter 2.

5.0 ANALYSIS OF PAVEMENT STRUCTURES CONTAINING ASPHALT TREATED PERMEABLE BASE

The Caltrans thickness design procedure is intended to provide an equivalent structural capacity of an asphalt concrete pavement if either an asphalt treated permeable base (ATPB) layer or an untreated aggregate base is included in the structure. In the Caltrans procedure, the structural strength of the ATPB layer in the pavement is a function of its thickness and its gravel equivalent factor (G_f). The thickness of the underlying aggregate base is reduced based on the ratio of the G_f values for ATPB and aggregate base. The current G_f for ATPB is 1.4 (based primarily on the judgment of Caltrans engineers) while the G_f for Class 2 aggregate base is 1.1 and that for the asphalt concrete ranges between 1.46 and 2.54.

5.1 Analysis Objectives

The original objective of this investigation was to perform simulations of pavement performance in order to develop a sound basis for selection of a gravel factor, or gravel factors, for ATPB material in the Caltrans pavement design procedure. A second objective was to quantify the effect of a reduction in the stiffness of the ATPB layer caused by soaking on predicted pavement fatigue life.

The investigation presented herein includes simulations that compare the performance that might be expected for the Caltrans pavements in the following three cases:

1. with Class 2 aggregate base and no ATPB layer,
2. with ATPB before it has experienced soaking, and
3. with ATPB after it has been subjected to periods of saturation.

Simulations of the change in stiffness of the ATPB caused by soaking are based on the laboratory tests described in Chapter 4 of this report.

5.2 Fatigue Analysis Procedure

The procedure used to estimate the performance of the pavement structures was initially developed under the SHRP program and modified during the CAL/APT program for Caltrans by the University of California, Berkeley Pavement Research Center. Its developments to date are summarized in References *(18)*, *(19)*, and *(17)*, the latter describing the results of the HVS test on drained pavement Section 500RF (associated with Goal 1 of the CAL/APT program).

5.2.1 Pavement Thickness Designs

Twenty-one asphalt concrete surfaced pavement cross-sections were designed using the Caltrans thickness design procedure for subgrade R-values of 5, 20, and 40; Traffic Indexes of 8, 9, 10, 11, 12, 13, and 14; and incorporating untreated aggregate. Both Class 2 aggregate subbase and Class 2 aggregate base were included in the designs, except that no subbase was used when the subgrade R-value was equal to 40. The minimum aggregate base thickness permitted was 152 mm. The computer program NEWCON90 was used for all the designs.

An additional 63 pavement structures were designed by incorporating a 75-mm thick layer of ATPB between the asphalt concrete and aggregate base and reducing the thickness of the aggregate base layer based on the ratios of ATPB gravel factors of 1.4, 1.9, and 1.1 and the aggregate base gravel factor of 1.1.

The resulting pavement structures containing only untreated aggregate are listed in Table 5.1. Also shown in the table are the stiffnesses and Poisson's ratios for each structure used to determine the critical tensile strain in the asphalt concrete and the vertical compressive strain at the subgrade surface for control of surface rutting. The designed pavement structures containing ATPB layers are shown in Tables 5.2, 5.3, and 5.4 for ATPB gravel factors of 1.4, 1.1, and 1.9, respectively.

NEWCON90 typically provides many pavement cross-sections that meet the structural requirements for the given inputs. Selection of the set of layer thicknesses for analysis reported herein was based on the following criteria:

- use of the thinnest permissible asphalt concrete layer thickness, then of those,
- use of the section with the lowest cost, assuming the default unit costs of in-place materials included in NEWCON90.

5.2.2 Fatigue Life Calculation

The fatigue life for each design was calculated using the UCB procedure described in Reference *(18)*. The equivalent single axle loads (ESALs) to fatigue failure were calculated using the following equation:

$$ESALs = \frac{N \cdot SF}{TCF \cdot M} \tag{5.1}$$

Table 5.1 Undrained (Aggregate Base Only) Pavement Structures Designed using Caltrans Method and Elastic Layer Theory Properties.

		Asphalt Concrete					Class 2 Aggregate Base					Class 2 Aggregate Subbase					Subgrade						metric
Traffic	Subgrade	Stiffness		Thickness		Poisson's	Stiffness		Thickness		Poisson's	Stiffness		Thickness		Poisson's	Stiffness		Poisson's	desgn thicknesses (ft)			GE
Index	R-value	(MPa)	(ksi)	(m)	(ft)	Ratio	(MPa)	(ksi)	(m)	(ft)	Ratio	(MPa)	(ksi)	(m)	(ft)	Ratio	(MPa)	(ksi)	Ratio	ac	ab	asb	ab
8	5	6730	977	0.122	0.40	0.35	207	30	0.183	0.60	0.35	138	20	0.290	0.95	0.35	27	4	0.45	0.4	0.6	0.95	
	20	6730	977	0.122	0.40	0.35	207	30	0.183	0.60	0.35	138	20	0.183	0.60	0.35	84	12	0.45	0.4	0.6	0.6	
	40	6730	977	0.122	0.40	0.35	207	30	0.198	0.65	0.35						161	23	0.45	0.4	0.65	0	
9	5	6730	977	0.137	0.45	0.35	207	30	0.213	0.70	0.35	138	20	0.335	1.10	0.35	27	4	0.45	0.45	0.7	1.1	
	20	6730	977	0.137	0.45	0.35	207	30	0.213	0.70	0.35	138	20	0.213	0.70	0.35	84	12	0.45	0.45	0.7	0.7	
	40	6730	977	0.137	0.45	0.35	207	30	0.244	0.80	0.35						161	23	0.45	0.45	0.8	0	
10	5	6730	977	0.152	0.50	0.35	207	30	0.244	0.80	0.35	138	20	0.381	1.25	0.35	27	4	0.45	0.5	0.8	1.25	0.88
	20	6730	977	0.152	0.50	0.35	207	30	0.244	0.80	0.35	138	20	0.244	0.80	0.35	84	12	0.45	0.5	0.8	0.8	0.88
	40	6730	977	0.152	0.50	0.35	207	30	0.290	0.95	0.35						161	23	0.45	0.5	0.95	0	1.05
11	5	6730	977	0.168	0.55	0.35	207	30	0.274	0.90	0.35	138	20	0.427	1.40	0.35	27	4	0.45	0.55	0.9	1.4	0.99
	20	6730	977	0.168	0.55	0.35	207	30	0.274	0.90	0.35	138	20	0.259	0.85	0.35	84	12	0.45	0.55	0.9	0.85	0.99
	40	6730	977	0.168	0.55	0.35	207	30	0.320	1.05	0.35						161	23	0.45	0.55	1.05	0	1.16
12	5	6730	977	0.183	0.60	0.35	172	25	0.305	1.00	0.35	138	20	0.473	1.55	0.35	27	4	0.45	0.6	1	1.55	1.1
	20	6730	977	0.183	0.60	0.35	172	25	0.305	1.00	0.35	138	20	0.290	0.95	0.35	84	12	0.45	0.6	1	0.95	1.1
	40	6730	977	0.183	0.60	0.35	172	25	0.351	1.15	0.35						161	23	0.45	0.6	1.15	0	1.27
13	5	6730	977	0.198	0.65	0.35	172	25	0.335	1.10	0.35	138	20	0.503	1.65	0.35	27	4	0.45	0.65	1.1	1.65	1.21
	20	6730	977	0.198	0.65	0.35	172	25	0.335	1.10	0.35	138	20	0.305	1.00	0.35	84	12	0.45	0.65	1.1	1	1.21
	40	6730	977	0.198	0.65	0.35	172	25	0.396	1.30	0.35						161	23	0.45	0.65	1.3	0	1.43
14	5	6730	977	0.213	0.70	0.35	172	25	0.351	1.15	0.35	138	20	0.564	1.85	0.35	27	4	0.45	0.7	1.15	1.85	1.27
	20	6730	977	0.213	0.70	0.35	172	25	0.351	1.15	0.35	138	20	0.351	1.15	0.35	84	12	0.45	0.7	1.15	1.15	1.27
	40	6730	977	0.213	0.70	0.35	172	25	0.427	1.40	0.35						161	23	0.45	0.7	1.4	0	1.54

Table 5.2 **Drained (with ATPB) Pavement Structures Designed by Caltrans Method (ATPB Gravel Factor = 1.4) and Elastic Layer Theory Properties.**

		Asphalt Concrete				Asphalt Treated Permeable Base Gf = 1.4					Class 2 Aggregate Base				Class 2 Aggregate Subbase				Subgrade	
						Stiffness														
Traffic Index	Subgrade R-value	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	As-Compacted (MPa)	Soaked (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Poisson's Ratio
8	5	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.267	0.88	0.35	27	0.45
	20	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.160	0.53	0.35	84	0.45
	40	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.101	0.33	0.35					161	0.45
9	5	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.116	0.38	0.35	138	0.335	1.10	0.35	27	0.45
	20	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.116	0.38	0.35	138	0.213	0.70	0.35	84	0.45
	40	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.147	0.48	0.35					161	0.45
10	5	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.147	0.48	0.35	138	0.381	1.25	0.35	27	0.45
	20	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.147	0.48	0.35	138	0.244	0.80	0.35	84	0.45
	40	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.193	0.63	0.35					161	0.45
11	5	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.177	0.58	0.35	138	0.427	1.40	0.35	27	0.45
	20	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.177	0.58	0.35	138	0.259	0.85	0.35	84	0.45
	40	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.223	0.73	0.35					161	0.45
12	5	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.208	0.68	0.35	138	0.473	1.55	0.35	27	0.45
	20	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.208	0.68	0.35	138	0.290	0.95	0.35	84	0.45
	40	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.254	0.83	0.35					161	0.45
13	5	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.238	0.78	0.35	138	0.503	1.65	0.35	27	0.45
	20	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.238	0.78	0.35	138	0.305	1.00	0.35	84	0.45
	40	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.299	0.98	0.35					161	0.45
14	5	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.254	0.83	0.35	138	0.564	1.85	0.35	27	0.45
	20	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.254	0.83	0.35	138	0.351	1.15	0.35	84	0.45
	40	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.330	1.08	0.35					161	0.45

Table 5.3 Drained (with ATPB) Pavement Structures Designed by Caltrans Method (ATPB Gravel Factor = 1.1) and Elastic Layer Theory Properties.

		Asphalt Concrete				Asphalt Treated Permeable Base Gf = 1.1					Class 2 Aggregate Base				Class 2 Aggregate Subbase				Subgrade	
						Stiffness														
Traffic Index	Subgrade R-value	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	As-Compacted (MPa)	Soaked (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Poisson's Ratio
8	5	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.290	0.95	0.35	27	0.45
	20	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.183	0.60	0.35	84	0.45
	40	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.122	0.40	0.35					161	0.45
9	5	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.137	0.45	0.35	138	0.335	1.10	0.35	27	0.45
	20	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.137	0.45	0.35	138	0.213	0.70	0.35	84	0.45
	40	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.168	0.55	0.35					161	0.45
10	5	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.168	0.55	0.35	138	0.381	1.25	0.35	27	0.45
	20	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.168	0.55	0.35	138	0.244	0.80	0.35	84	0.45
	40	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.213	0.70	0.35					161	0.45
11	5	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.198	0.65	0.35	138	0.427	1.40	0.35	27	0.45
	20	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.198	0.65	0.35	138	0.259	0.85	0.35	84	0.45
	40	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.244	0.80	0.35					161	0.45
12	5	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.229	0.75	0.35	138	0.473	1.55	0.35	27	0.45
	20	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.229	0.75	0.35	138	0.290	0.95	0.35	84	0.45
	40	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.274	0.90	0.35					161	0.45
13	5	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.259	0.85	0.35	138	0.503	1.65	0.35	27	0.45
	20	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.259	0.85	0.35	138	0.305	1.00	0.35	84	0.45
	40	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.320	1.05	0.35					161	0.45
14	5	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.274	0.90	0.35	138	0.564	1.85	0.35	27	0.45
	20	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.274	0.90	0.35	138	0.351	1.15	0.35	84	0.45
	40	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.351	1.15	0.35					161	0.45

Table 5.4 Drained (with ATPB) Pavement Structures Designed by Caltrans Method (ATPB Gravel Factor = 1.9) and Elastic Layer Theory Properties.

		Asphalt Concrete				Asphalt Treated Permeable Base Gf = 1.9					Class 2 Aggregate Base				Class 2 Aggregate Subbase				Subgrade	
						Stiffness														
Traffic Index	Subgrade R-value	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	As-Compacted (MPa)	Soaked (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Thickness (m)	Thickness (ft)	Poisson's Ratio	Stiffness (MPa)	Poisson's Ratio
8	5	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.229	0.75	0.35	27	0.45
	20	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.122	0.40	0.35	84	0.45
	40	6730	0.122	0.40	0.35	1172	500	0.076	0.25	0.4	172	0.067	0.22	0.35					161	0.45
9	5	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.308	1.01	0.35	27	0.45
	20	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.107	0.35	0.35	138	0.186	0.61	0.35	84	0.45
	40	6730	0.137	0.45	0.35	1172	500	0.076	0.25	0.4	172	0.112	0.37	0.35					161	0.45
10	5	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.112	0.37	0.35	138	0.381	1.25	0.35	27	0.45
	20	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.112	0.37	0.35	138	0.244	0.80	0.35	84	0.45
	40	6730	0.152	0.50	0.35	1172	500	0.076	0.25	0.4	172	0.158	0.52	0.35					161	0.45
11	5	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.143	0.47	0.35	138	0.427	1.40	0.35	27	0.45
	20	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.143	0.47	0.35	138	0.259	0.85	0.35	84	0.45
	40	6730	0.168	0.55	0.35	1172	500	0.076	0.25	0.4	172	0.188	0.62	0.35					161	0.45
12	5	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.173	0.57	0.35	138	0.473	1.55	0.35	27	0.45
	20	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.173	0.57	0.35	138	0.290	0.95	0.35	84	0.45
	40	6730	0.183	0.60	0.35	1172	500	0.076	0.25	0.4	172	0.219	0.72	0.35					161	0.45
13	5	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.204	0.67	0.35	138	0.503	1.65	0.35	27	0.45
	20	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.204	0.67	0.35	138	0.305	1.00	0.35	84	0.45
	40	6730	0.198	0.65	0.35	1172	500	0.076	0.25	0.4	172	0.265	0.87	0.35					161	0.45
14	5	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.219	0.72	0.35	138	0.564	1.85	0.35	27	0.45
	20	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.219	0.72	0.35	138	0.351	1.15	0.35	84	0.45
	40	6730	0.213	0.70	0.35	1172	500	0.076	0.25	0.4	172	0.295	0.97	0.35					161	0.45

where $ESALs$ = equivalent single axle loads to fatigue cracking,

N = laboratory fatigue life from flexural beam test results,

SF = laboratory to field shift factor to account for traffic wander and crack propagation,

TCF = temperature conversion factor to account for differences in asphalt concrete stiffness and fatigue life between the single laboratory test temperature and the range of temperatures occurring in the field, and

M = reliability multiplier which includes estimates of the variance from construction and laboratory test variables.

The temperature environment utilized was that for the California coastal region of Santa Barbara. The temperature conversion factor for this environment has already been reported. *(17, 18)*

A "standard" asphalt concrete mix was assumed, described in Reference *(16)*. The standard mix contains an AR-4000 grade asphalt cement from California Valley sources and a crushed granite from Watsonville, California. The gradation passes between the coarse and medium gradations of the Caltrans specifications for 19-mm maximum aggregate size, meeting the requirements for both gradations.

The asphalt content was assumed to be 5 percent, approximately the asphalt content that would be obtained for a Type A asphalt concrete. The air-void content was assumed to be 8 percent, which corresponds to approximately 97-percent compaction relative to the laboratory test maximum density according to Caltrans Test Method 304.

The stiffness and laboratory fatigue life relation of the standard asphalt concrete mix corresponded to laboratory test results previously published in Reference *(18)*. For the assumed asphalt content and air-void content, the asphalt concrete stiffness is 6,730 MPa. The laboratory fatigue life relation is:

$$N = 2.7867 \times 10^{-10} e^{-0.165AV+0.575AC} \times \varepsilon_t^{-3.78} \tag{5.2}$$

where N = laboratory fatigue life,

AV = percent air-voids,

AC = percent asphalt content by mass of aggregate, and

ε_t = tensile strain.

The assumed stiffnesses of the ATPB as-compacted and after soaking were based on the laboratory testing described in Chapter 4 of this report. Stiffnesses of 1172 and 500 MPa were assumed to be representative of as-compacted and soaked conditions, respectively. Calculations of stresses in the ATPB layer under a standard 80-kN single axle load using layered elastic analysis typically indicate horizontal tensile stresses in situ, while compressive stresses are usually applied during laboratory triaxial testing. For the analyses reported herein, representative stiffnesses are based on the laboratory results from triaxial test specimens subjected to small confining stresses; these are shown in Figure 5.1.

Moduli used for the aggregate base, aggregate subbase, and subgrade are based on those used in the calibration of the UCB fatigue analysis procedure to California conditions *(19)*. The subgrade moduli corresponding to the three R-values included in the experiment design were assumed to be:

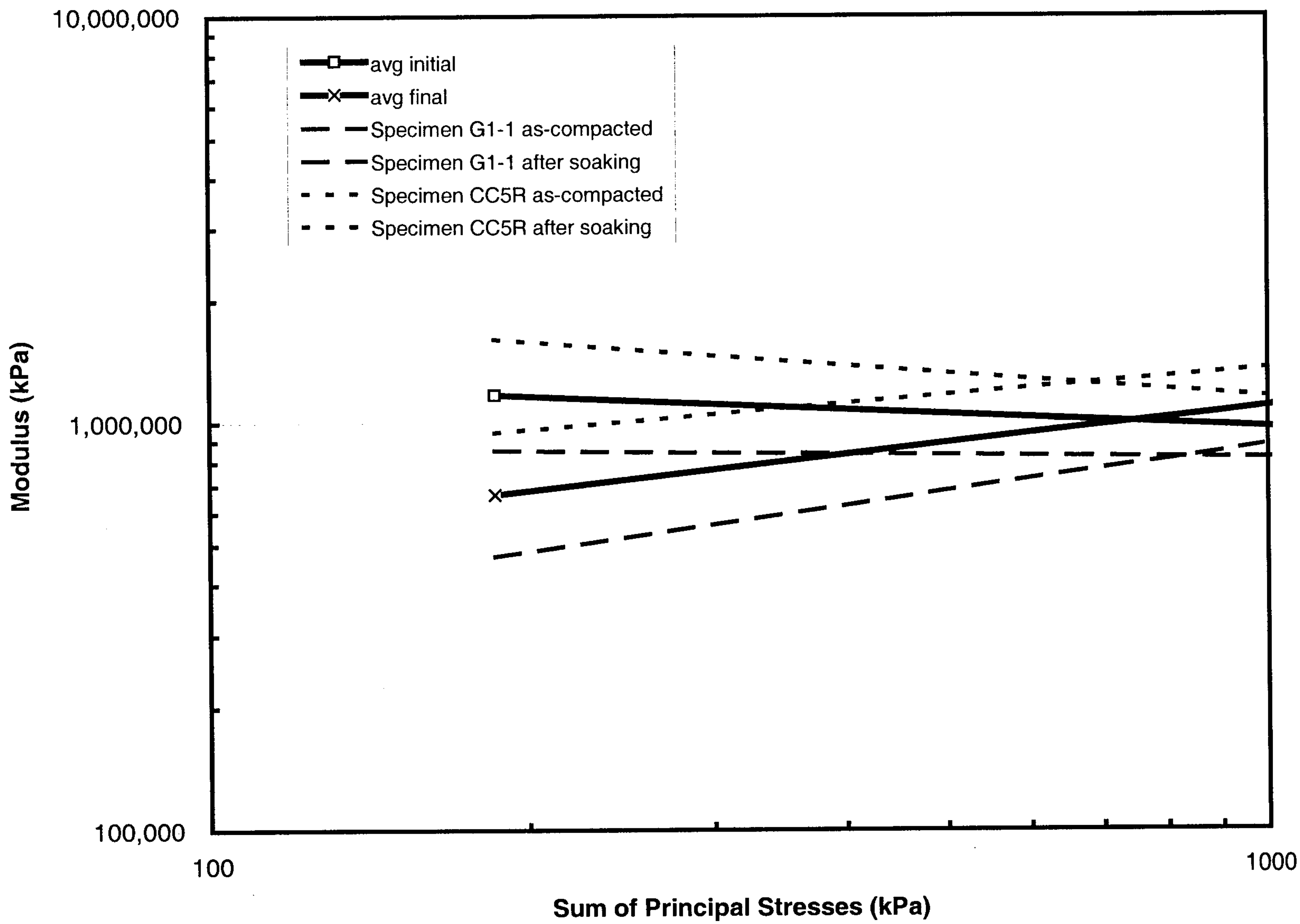

Figure 5.1. Resilient Modulus Results Used to Select Representative As-Compacted and Soaked Elastic Moduli for ATPB.

Subgrade R-value	Modulus (MPa)
5	27
20	84
40	161

The aggregate subbase was assumed to have a modulus of 138 MPa. The aggregate base modulus was assumed to be related to the thickness of the asphalt-bound layers above it, as follows:

Asphalt Concrete + ATPB Thickness (mm)	Aggregate Base Modulus (MPa)
≤ 183	207
≤ 305	172
> 305	138

The critical strains for fatigue cracking and rutting of the unbound layers were calculated using CIRCLY. *(20)* A uniform contact pressure with no surface shearing forces at one surface and a standard 80-kN single axle load were assumed. A tire contact pressure of 690 kPa was utilized, with the dual wheels spaced 317.5 mm (12.5 in.) apart.

The critical strain for fatigue cracking was assumed to be the largest tensile principal strain occurring at the bottom of the asphalt concrete layer. Strains were evaluated under one of the tires, at the mid-point between the two tires, and at the outside edge of one of the tires.

5.3 Fatigue Analysis Results

5.3.1 Simulated Pavement Fatigue Life

The simulated ESALs to fatigue cracking were compared with the Caltrans equation relating traffic index to ESALs:

$$ESALs = 1.2895 \times 10^{-2} TI^{8.2919} \tag{5.3}$$

as can be seen in Figures 5.2 and 5.3. Simulated pavement fatigue lives do not include a factor for reliability.

In Figure 5.2 the simulated fatigue lives of the pavements containing as-compacted ATPB can be seen to be significantly larger than those of the pavements containing aggregate base only. This would be expected, based on the ratio of the stiffnesses of the ATPB and aggregate base and the ratio at which the thickness of the aggregate base layer is adjusted for inclusion of ATPB in the structure. The as-compacted stiffness of the ATPB is about 6.9 times greater than that of the aggregate base, while aggregate base is replaced by ATPB in the pavement at a ratio of 1:1.27 when the gravel factor for ATPB is 1.4.

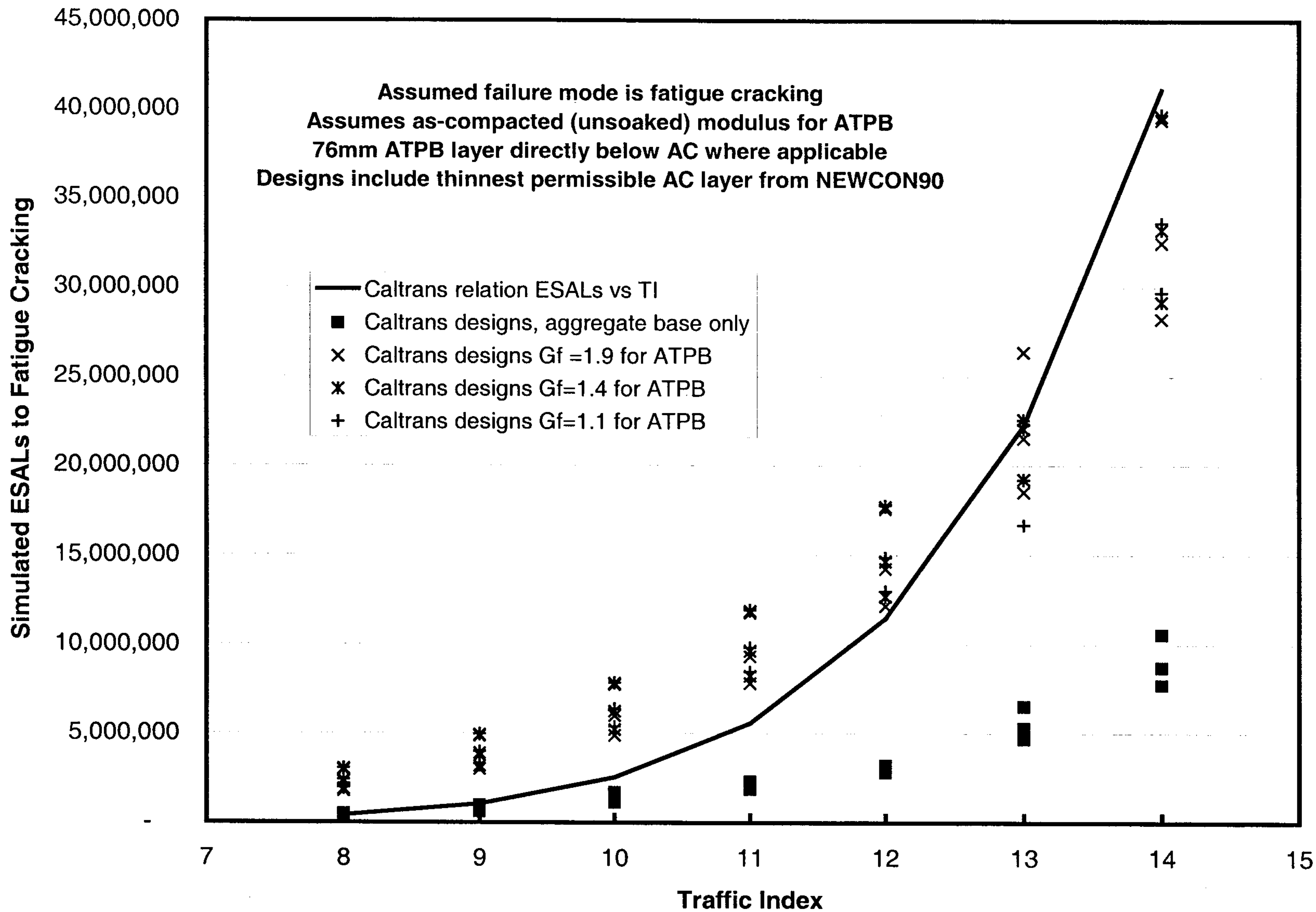

Figure 5.2. Comparison of Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Simulated Pavement Fatigue Life for Pavements with As-Compacted ATPB and with Aggregate Base Only.

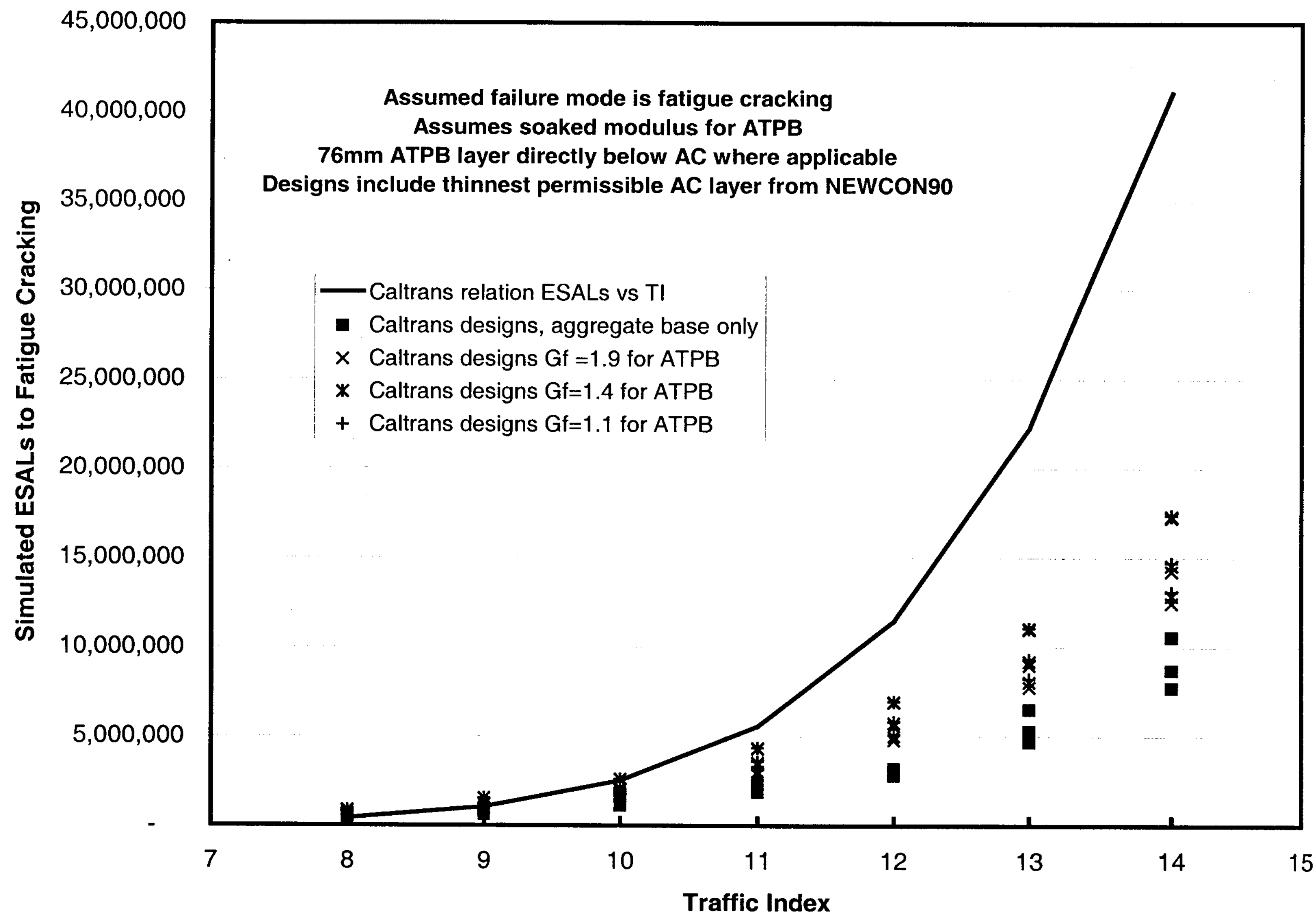

Figure 5.3. Comparison of Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Simulated Pavement Fatigue Life for Pavements with Soaked ATPB and with Aggregate Base Only.

When evaluated for allowable number of ESALs using the UCB fatigue analysis procedure and comparing that number to the allowable ESALs assumed by the Caltrans design method, the Caltrans method overdesigns for TIs of 12 or less and generally underdesigns for TIs of 13 and 14.

In Figure 5.3 it can be seen that the simulated fatigue lives of the pavements containing ATPB that has been soaked for 10 days are considerably reduced compared to those containing as-compacted ATPB, although they are still greater than those of the pavements that do not include ATPB. The stiffness of the soaked ATPB is about 2.9 times greater than that of the aggregate base. The simulated fatigue lives for the soaked ATPB pavements are generally less than the Caltrans design relation for TIs of 10 or more. The difference between the simulated fatigue lives and the Caltrans design relation is greater for larger traffic indices.

5.3.2 Design Pavement Fatigue Life

An appropriate factor for reliability should be included in computations of pavement fatigue life for design purposes. The design fatigue lives of the pavements assuming a reliability of 90 percent compared to the Caltrans design assumption of ESALs versus traffic index are shown in Figures 5.4 and 5.5.

Ratios of design fatigue life and simulated fatigue life versus the Caltrans design relation for ESALs versus traffic index were calculated. They are shown in Table 5.5 and 5.6 for the as-compacted and soaked ATPB cases, respectively. The ratios for the aggregate base pavements are also included in both tables. A ratio of 1.0 or greater indicates that the predicted pavement

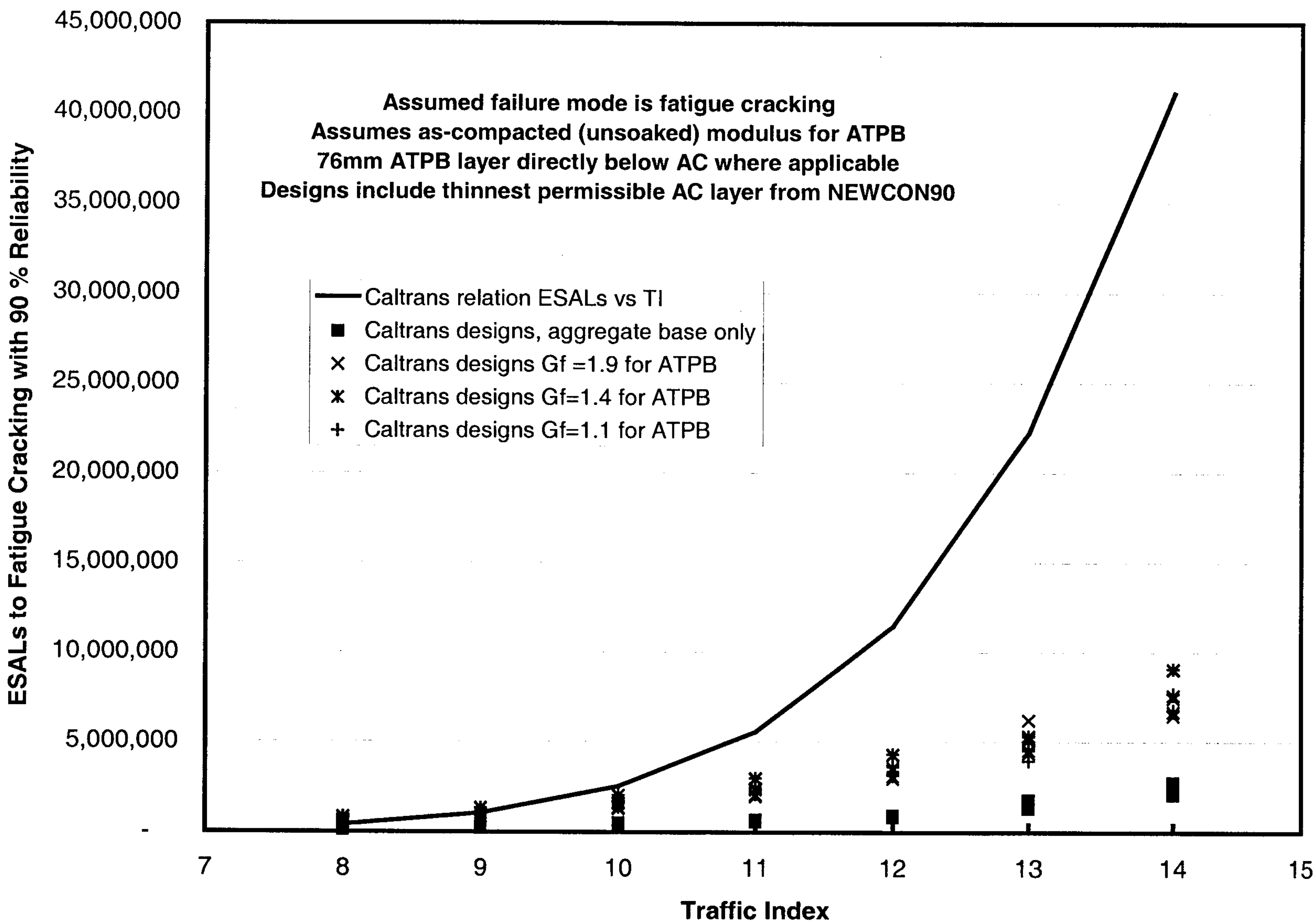

Figure 5.4. Comparison of Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Pavement Fatigue Life with 90 Percent Reliability for Pavements with As-Compacted ATPB and with Aggregate Base Only.

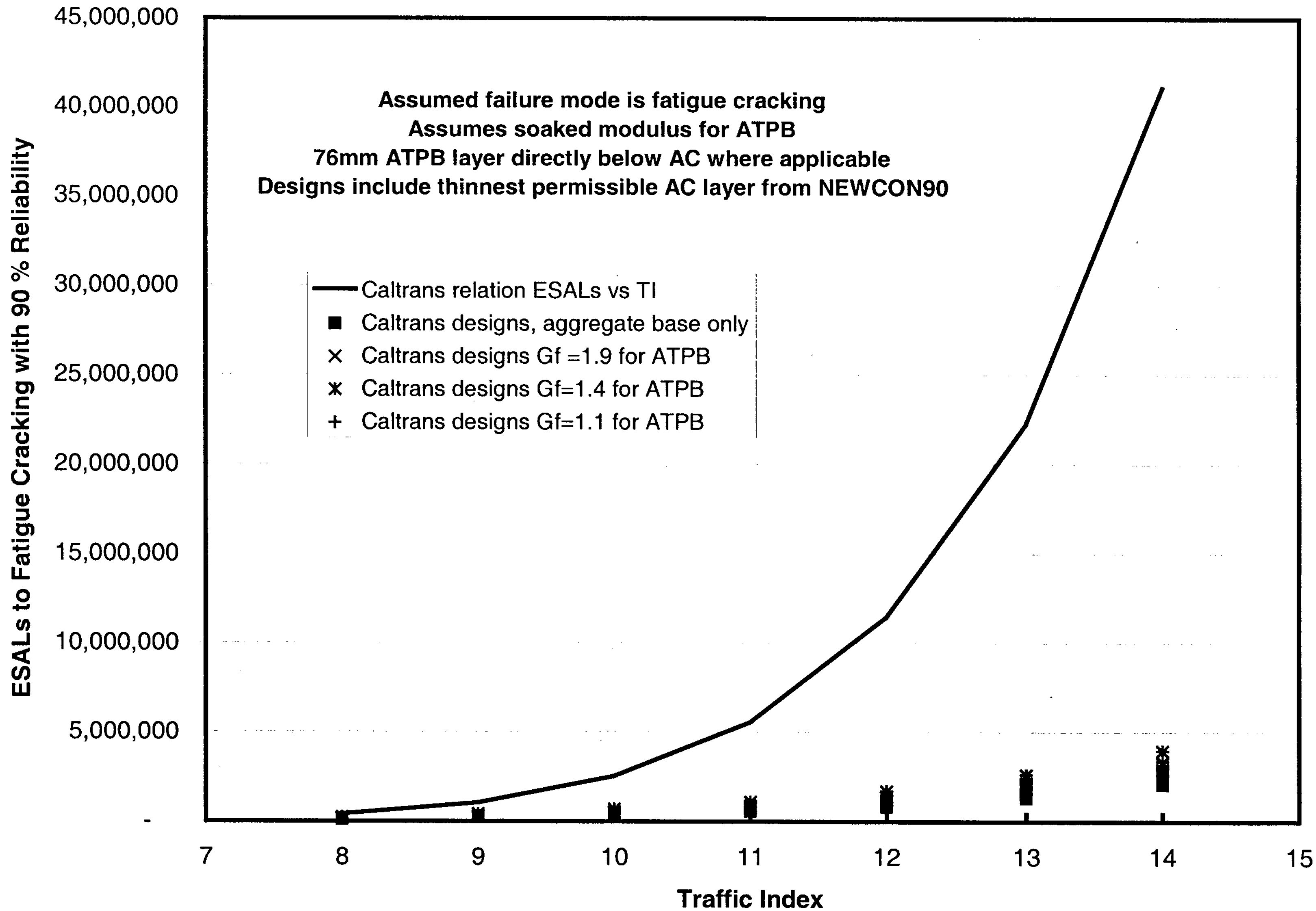

Figure 5.5. Comparison of Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Pavement Fatigue Life with 90 Percent Reliability for Pavements with Soaked ATPB and with Aggregate Base Only.

Table 5.5 Ratio of Simulated and 90 Percent Reliability Pavement Fatigue Life (ESALs) to Caltrans Design ESALs for Undrained Pavements and Drained Pavements with As-Compacted ATPB.

Traffic Index	Subgrade R-value	Caltrans Design ESALs	Ratio of Pavement Fatigue Life (soaked ATPB) / Caltrans Design ESALs — Undrained, Aggregate Base Only: Simulated	Undrained, Aggregate Base Only: 90 % Reliability	Drained, ATPB Gf = 1.9: Simulated	Drained, ATPB Gf = 1.9: 90 % Reliability	Drained, ATPB Gf = 1.4: Simulated	Drained, ATPB Gf = 1.4: 90 % Reliability	Drained, ATPB Gf = 1.1: Simulated	Drained, ATPB Gf = 1.1: 90 % Reliability
8	5	396,966	0.73	0.24	1.23	0.34	1.32	0.37	1.37	0.38
8	20	396,966	0.90	0.29	1.60	0.44	1.66	0.46	1.69	0.47
8	40	396,966	1.23	0.39	2.10	0.58	2.14	0.59	2.16	0.60
9	5	1,054,156	0.57	0.18	0.89	0.24	0.95	0.25	0.99	0.26
9	20	1,054,156	0.68	0.21	1.12	0.30	1.15	0.31	1.18	0.32
9	40	1,054,156	0.90	0.28	1.45	0.39	1.47	0.39	1.48	0.40
10	5	2,525,349	0.44	0.13	0.65	0.17	0.69	0.18	0.72	0.19
10	20	2,525,349	0.51	0.16	0.80	0.21	0.83	0.22	0.85	0.22
10	40	2,525,349	0.67	0.20	1.03	0.27	1.04	0.27	1.04	0.27
11	5	5,566,028	0.36	0.11	0.52	0.13	0.55	0.14	0.56	0.14
11	20	5,566,028	0.41	0.12	0.61	0.15	0.63	0.16	0.65	0.16
11	40	5,566,028	0.34	0.10	0.78	0.19	0.78	0.20	0.79	0.20
12	5	11,452,079	0.24	0.07	0.42	0.10	0.44	0.11	0.45	0.11
12	20	11,452,079	0.28	0.08	0.49	0.12	0.50	0.12	0.51	0.12
12	40	11,452,079	0.27	0.08	0.60	0.15	0.61	0.15	0.61	0.15
13	5	22,239,664	0.21	0.06	0.35	0.08	0.36	0.09	0.37	0.09
13	20	22,239,664	0.24	0.07	0.40	0.10	0.41	0.10	0.42	0.10
13	40	22,239,664	0.29	0.08	0.49	0.12	0.50	0.12	0.50	0.12
14	5	41,115,011	0.19	0.05	0.30	0.07	0.31	0.07	0.32	0.07
14	20	41,115,011	0.21	0.06	0.35	0.08	0.35	0.08	0.36	0.08
14	40	41,115,011	0.26	0.07	0.42	0.10	0.42	0.10	0.42	0.10

Table 5.6 Ratio of Simulated and 90 Percent Reliability Pavement Fatigue Life (ESALs) to Caltrans Design ESALs for Undrained Pavements and Drained Pavements with Soaked ATPB.

Traffic Index	Subgrade R-value	Caltrans Design ESALs	Ratio of Pavement Fatigue Life (as-compacted ATPB) / Caltrans Design ESALs							
			Undrained		Drained					
			Aggregate Base Only		ATPB Gf = 1.9		ATPB Gf = 1.4		ATPB Gf = 1.1	
			Simulated	90 % Reliability	Simulated	90 % Reliability	Simulated	90 % Reliability	Simulated	90 % Reliability
8	5	396,966	0.73	0.24	4.41	1.22	4.72	1.31	4.90	1.36
8	20	396,966	0.90	0.29	5.74	1.59	5.94	1.65	6.06	1.68
8	40	396,966	1.23	0.39	7.44	2.06	7.60	2.11	7.67	2.13
9	5	1,054,156	0.57	0.18	2.84	0.76	3.01	0.81	3.13	0.84
9	20	1,054,156	0.68	0.21	3.57	0.96	3.68	0.99	3.78	1.01
9	40	1,054,156	0.90	0.28	4.61	1.24	4.67	1.25	4.70	1.26
10	5	2,525,349	0.44	0.13	1.94	0.50	2.06	0.54	2.14	0.55
10	20	2,525,349	0.51	0.16	2.37	0.62	2.47	0.64	2.53	0.66
10	40	2,525,349	0.67	0.20	3.06	0.79	3.09	0.80	3.11	0.81
11	5	5,566,028	0.36	0.11	1.41	0.35	1.48	0.37	1.52	0.38
11	20	5,566,028	0.41	0.12	1.67	0.42	1.73	0.43	1.76	0.44
11	40	5,566,028	0.34	0.10	2.11	0.53	2.13	0.53	2.14	0.54
12	5	11,452,079	0.24	0.07	1.06	0.26	1.11	0.27	1.13	0.28
12	20	11,452,079	0.28	0.08	1.24	0.30	1.28	0.31	1.30	0.32
12	40	11,452,079	0.27	0.08	1.54	0.37	1.55	0.38	1.55	0.38
13	5	22,239,664	0.21	0.06	0.83	0.20	0.87	0.20	0.75	0.18
13	20	22,239,664	0.24	0.07	0.97	0.23	0.99	0.23	0.86	0.20
13	40	22,239,664	0.29	0.08	1.19	0.28	1.02	0.24	1.01	0.24
14	5	41,115,011	0.19	0.05	0.69	0.16	0.71	0.16	0.72	0.17
14	20	41,115,011	0.21	0.06	0.79	0.18	0.81	0.19	0.82	0.19
14	40	41,115,011	0.26	0.07	0.96	0.22	0.96	0.22	0.97	0.22

fatigue life is adequate with respect to the Caltrans design relation. A ratio of less than 1.0 indicates that the predicted pavement fatigue life is less than is assumed by the Caltrans design relation; accordingly, the pavement would be expected to fail under a smaller number of ESALs than that for which it was designed.

The design pavement fatigue lives are generally adequate for traffic indices of 8 and 9 when the as-compacted stiffness of the ATPB is included in the pavement model, as can be seen in Figure 5.4 and Table 5.5. They are inadequate for traffic indices of 10 through 14. The difference between the Caltrans design relation and the design pavement fatigue life increases for larger traffic indices.

When the pavement includes the soaked stiffness of the ATPB, or when only aggregate base is used, the design pavement fatigue lives are less than the Caltrans design relation for all traffic indices included in the analysis, as can be seen in Figure 5.5 and Table 5.6.

5.4 Analysis Procedure for Rutting of the Unbound Layers

The ability of the pavement structures to resist rutting resulting from permanent deformation in the unbound layers was evaluated using the Asphalt Institute criteria, which are based on the vertical compressive strain at the top of the subgrade. *(21)* The relationship for subgrade strain is:

$$N = 1.05 \times 10^{-9} \varepsilon_c^{-4.484} \tag{5.4}$$

where N = number of load applications, and

ε_c = vertical compressive strain at subgrade surface.

The equation, first used by Santucci, is based on analyses of pavements designed according to the Caltrans pavement design procedure. *(22, 23)* The authors of the Asphalt Institute methodology state, *"if good compaction of the pavement components is obtained and the asphalt mix is well designed, rutting should not exceed about 12.7 mm (0.5 in.) at the surface for the design traffic, N."* This statement implies some conservatism in the criterion, but it does not include an explicit factor of safety or reliability estimate.

The allowable ESALs for the pavement structures containing as-compacted ATPB are shown plotted against the Caltrans design relation in Figure 5.6. A similar plot for the pavement structures containing soaked ATPB is included in Figure 5.7. The ratios of the allowable ESALs versus the Caltrans design relation are included in Tables 5.7 and 5.8 for the as-compacted and soaked ATPB pavements, respectively. The aggregate base pavements are included in each figure and table.

The results indicate that all of the pavements included in the analysis are adequate with regard to rutting of the unbound layers. Pavement structures that include subgrades with an R-value of 5 have a smaller, although adequate, resistance to rutting compared to those with subgrade R-values of 20 and 40.

It is apparent that the critical failure mode for the pavement structures included in this study is fatigue cracking, not rutting of the unbound materials.

5.5 Gravel Factor Determinations for ATPB

To ascertain gravel factors (G_f) that might be used by Caltrans, a series of analyses like those reported in the previous section were performed on pavements containing untreated

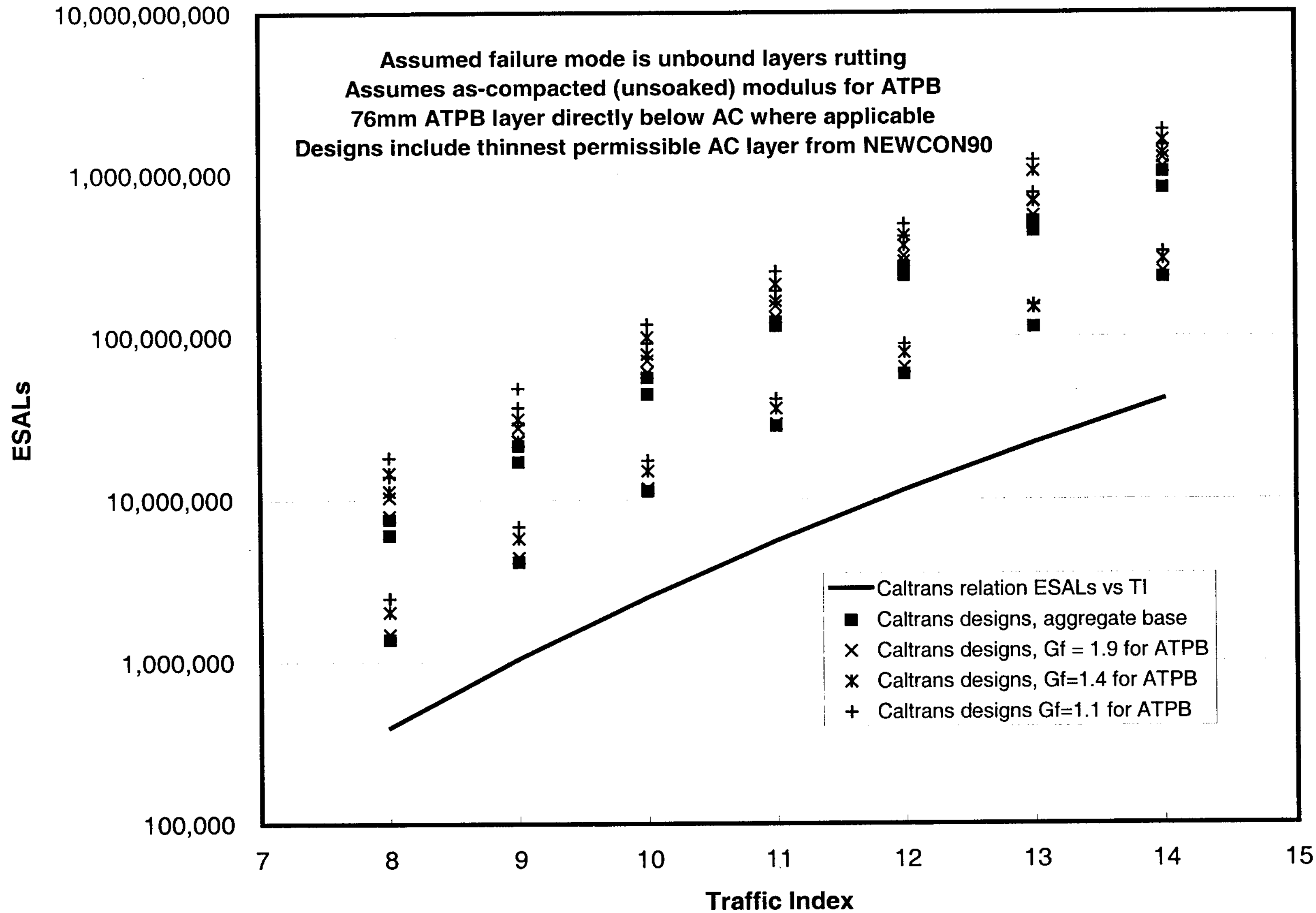

Figure 5.6. Comparison of the Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Design Pavement Rutting Life for Pavements with As-Compacted ATPB and with Aggregate Base Only (Rutting in Unbound Materials, Asphalt Institute Criterion *[21]*).

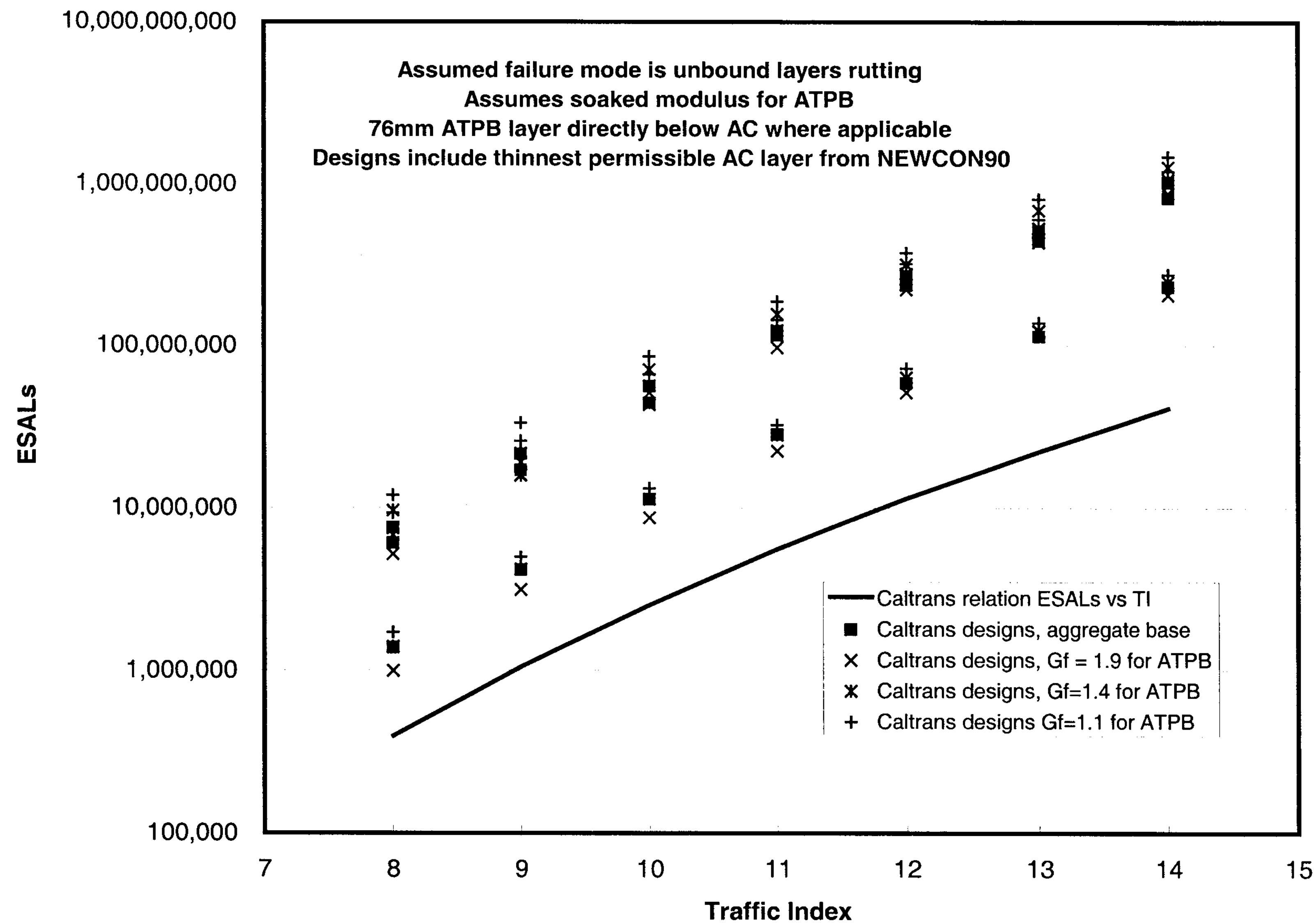

Figure 5.7. Comparison of the Caltrans Traffic Index (TI) versus Equivalent Single Axle Loads (ESALs) Relation and Design Pavement Rutting Life for Pavements with Soaked ATPB and with Aggregate Base Only (Rutting in Unbound Materials, Asphalt Institute Criterion *[21]*).

Table 5.7 **Ratio of Asphalt Institute Criterion *(21)* Design ESALs to Pavement Rutting Life (ESALs) to Caltrans Design ESALs for Undrained Pavements and Drained Pavements with As-Compacted ATPB.**

Traffic Index	Subgrade R-value	Caltrans Design ESALs	Ratio of ESALs to Unbound Layers Rutting (as-compacted ATPB) / Caltrans Design ESALs: Aggregate Base Only Simulated	ATPB Gf = 1.9 Simulated	ATPB Gf = 1.4 Simulated	ATPB Gf = 1.1 Simulated
8	5	396,966	3.47	3.74	5.14	6.21
8	20	396,966	19.03	20.00	28.41	34.96
8	40	396,966	15.21	26.09	36.75	45.54
9	5	1,054,156	3.92	4.19	5.50	6.47
9	20	1,054,156	20.33	21.91	29.55	34.97
9	40	1,054,156	16.15	26.23	21.65	45.80
10	5	2,525,349	4.46	4.64	5.95	6.88
10	20	2,525,349	22.20	23.89	30.99	36.13
10	40	2,525,349	17.55	28.24	39.31	47.29
11	5	5,566,028	5.09	5.22	6.51	7.44
11	20	5,566,028	22.08	23.53	29.73	34.33
11	40	5,566,028	20.79	27.80	37.80	45.35
12	5	11,452,079	5.13	5.66	6.98	7.88
12	20	11,452,079	23.35	25.37	31.66	36.06
12	40	11,452,079	20.54	27.31	36.47	42.87
13	5	22,239,664	5.13	6.83	6.79	7.06
13	20	22,239,664	22.96	24.87	30.43	34.29
13	40	22,239,664	19.96	30.69	46.80	54.44
14	5	41,115,011	5.61	6.06	7.27	8.05
14	20	41,115,011	24.99	26.78	32.48	36.22
14	40	41,115,011	19.89	30.39	39.12	45.34

Table 5.8 Ratio of Asphalt Institute Criterion *(21)* Design ESALs to Pavement Rutting Life (ESALs) to Caltrans Design ESALs for Undrained Pavements and Drained Pavements with Soaked ATPB.

Traffic Index	Subgrade R-value	Caltrans Design ESALs	Ratio of Pavement Fatigue Life (soaked ATPB) / Caltrans Design ESALs			
			Aggregate Base Only Simulated	ATPB Gf = 1.9 Simulated	ATPB Gf = 1.4 Simulated	ATPB Gf = 1.1 Simulated
8	5	396,966	3.47	2.51	3.52	4.30
8	20	396,966	19.03	13.01	18.74	23.26
8	40	396,966	15.21	17.13	24.19	30.05
9	5	1,054,156	3.92	2.97	3.97	4.71
9	20	1,054,156	20.33	15.04	20.53	24.46
9	40	1,054,156	16.15	18.11	14.93	31.64
10	5	2,525,349	4.46	3.45	4.47	5.21
10	20	2,525,349	22.20	17.11	22.42	26.31
10	40	2,525,349	17.55	20.27	28.17	33.89
11	5	5,566,028	5.09	4.02	5.06	5.81
11	20	5,566,028	22.08	17.50	22.27	25.79
11	40	5,566,028	20.79	20.56	28.05	33.57
12	5	11,452,079	5.13	4.48	5.56	6.32
12	20	11,452,079	23.35	19.35	24.33	27.90
12	40	11,452,079	20.54	20.75	27.70	32.61
13	5	22,239,664	5.13	5.56	5.53	6.23
13	20	22,239,664	22.96	19.43	23.89	27.09
13	40	22,239,664	19.96	23.80	30.82	36.13
14	5	41,115,011	5.61	5.00	6.02	6.69
14	20	41,115,011	24.99	21.35	26.07	29.16
14	40	41,115,011	19.89	24.03	30.95	35.90

aggregate base and pavements in which a thickness of aggregate base was replaced by 76 mm of ATPB according to the ratio of the gravel factors for the two materials. In the previous section, values for G_f for ATPB were the range 1.1 to 1.9. In these analyses, values of G_f for the ATPB were increased to as much as 4.5.

The analyses included pavement structures designed for the following conditions:

Traffic Index, TI: 10, 11, 12, 13, 14

R value, subgrade: 5, 20, 40

These resulting structures were subjected to the same load on dual tires as those described in Section 5.2.2. Tensile strains on the underside of the asphalt concrete and vertical compressive strain at the subgrade surface were computed using the CIRCLY program.

For the drained pavements (i.e., those containing ATPB) six values of the G_f for the ATPB were used for each TI and subgrade R-value combination. Values of the moduli of the asphalt concrete, aggregate base, aggregate subbase, and subgrade were the same as those described in Section 5.2.2. Two modulus values were used for the ATPB: as-compacted, 1,172 MPa; and soaked, 500 MPa.

To arrive at the gravel factors, both the tensile strain in the asphalt concrete and vertical compressive strain at the subgrade were utilized. The thickness of aggregate base in the drained section that produced the same tensile and vertical compressive strains as those calculated in the undrained pavement section was then determined. The G_f was then determined from the following expressions:

$$G_{f\ ATPB} = \frac{G_{f\ AB}\left(T_{AB\ _{ndrained}} - T_{AB\ _{rained}}\right)}{T_{ATPB}} \tag{5.5}$$

where:

$G_{f\ ATPB}$	=	gravel factor ATPB
$G_{f\ AB}$	=	gravel factor asphalt base (1.1)
$T_{AB_{undrained}}$	=	thickness of aggregate base in undrained section
$T_{AB_{drained}}$	=	thickness of aggregate base in drained section corresponding to the same tensile and vertical compressive strains in the undrained section

Representative results for the CIRCLY computations are plotted in Figures 5.8 through 5.13 for TIs of 10, 12, and 14 and subgrade R-values of 5 and 40. In these figures it will be noted that the tensile strain in the asphalt concrete is not very sensitive to $G_{f_{ATPB}}$. From the information plotted in this way, the values for determined from equation (5.5) are summarized in Table 5.9.

From an examination of this table it will be noted that while the values of $G_{f_{ATPB}}$ are high and variable based on considerations of fatigue, there are two relatively consistent values based on subgrade strain.

For the ATPB in the as-compacted condition an average value for $G_{f_{ATPB}}$ is about 2.2. For the material in the soaked conditions (10 days at 20°C in this instance) the average value is 1.7.

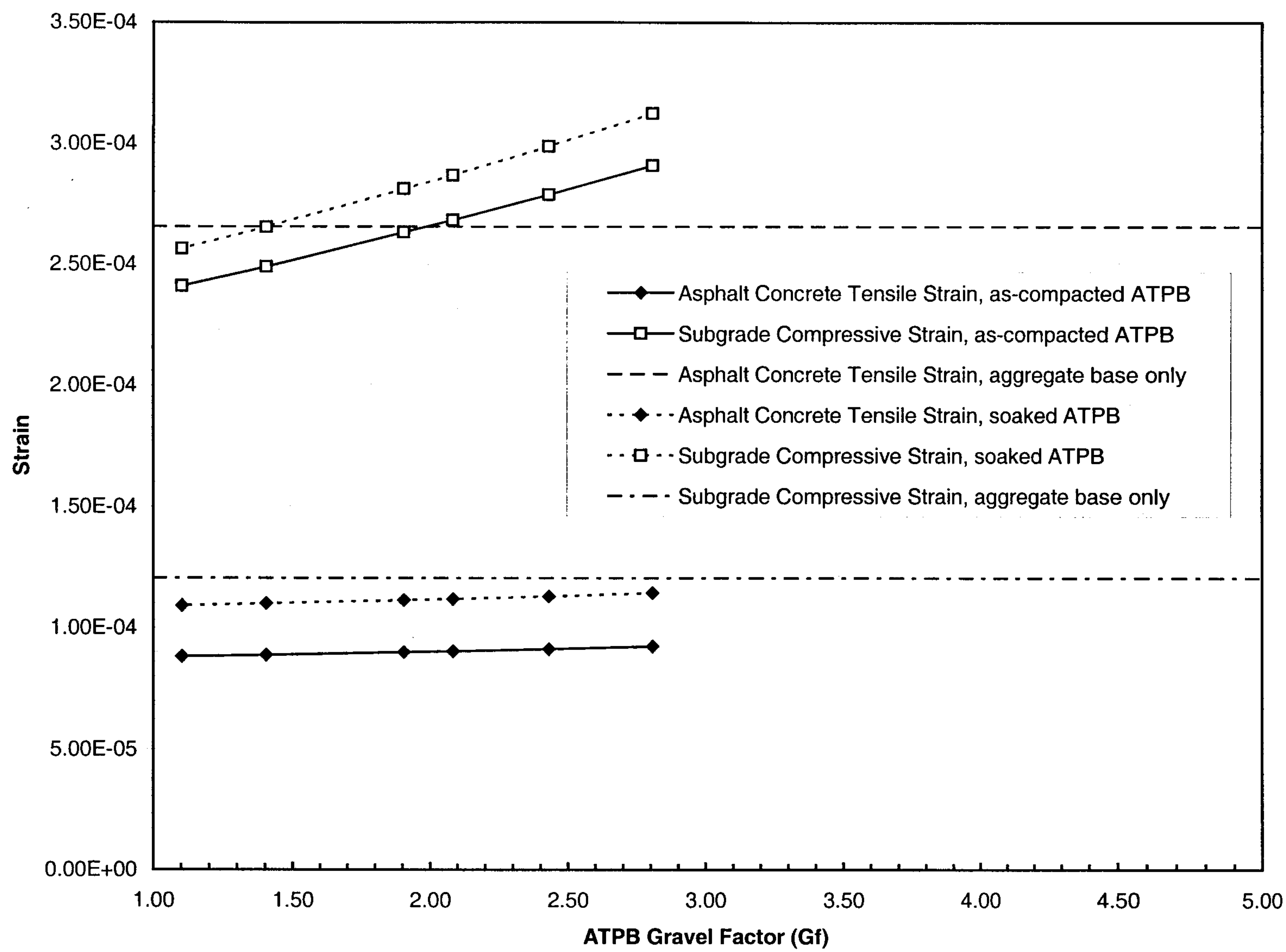

Figure 5.8. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=10, R-value=5).

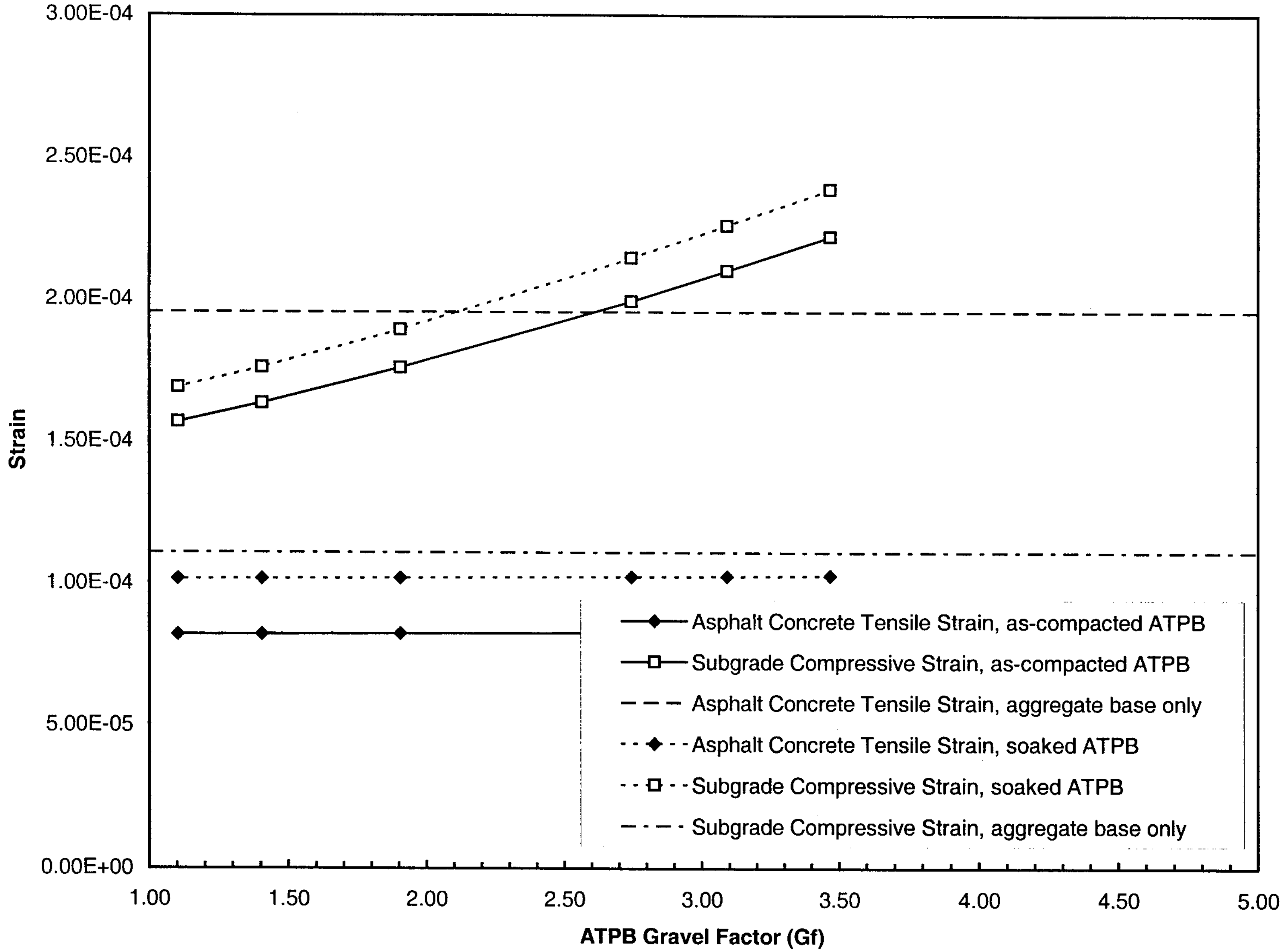

Figure 5.9. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=10, R-value=40).

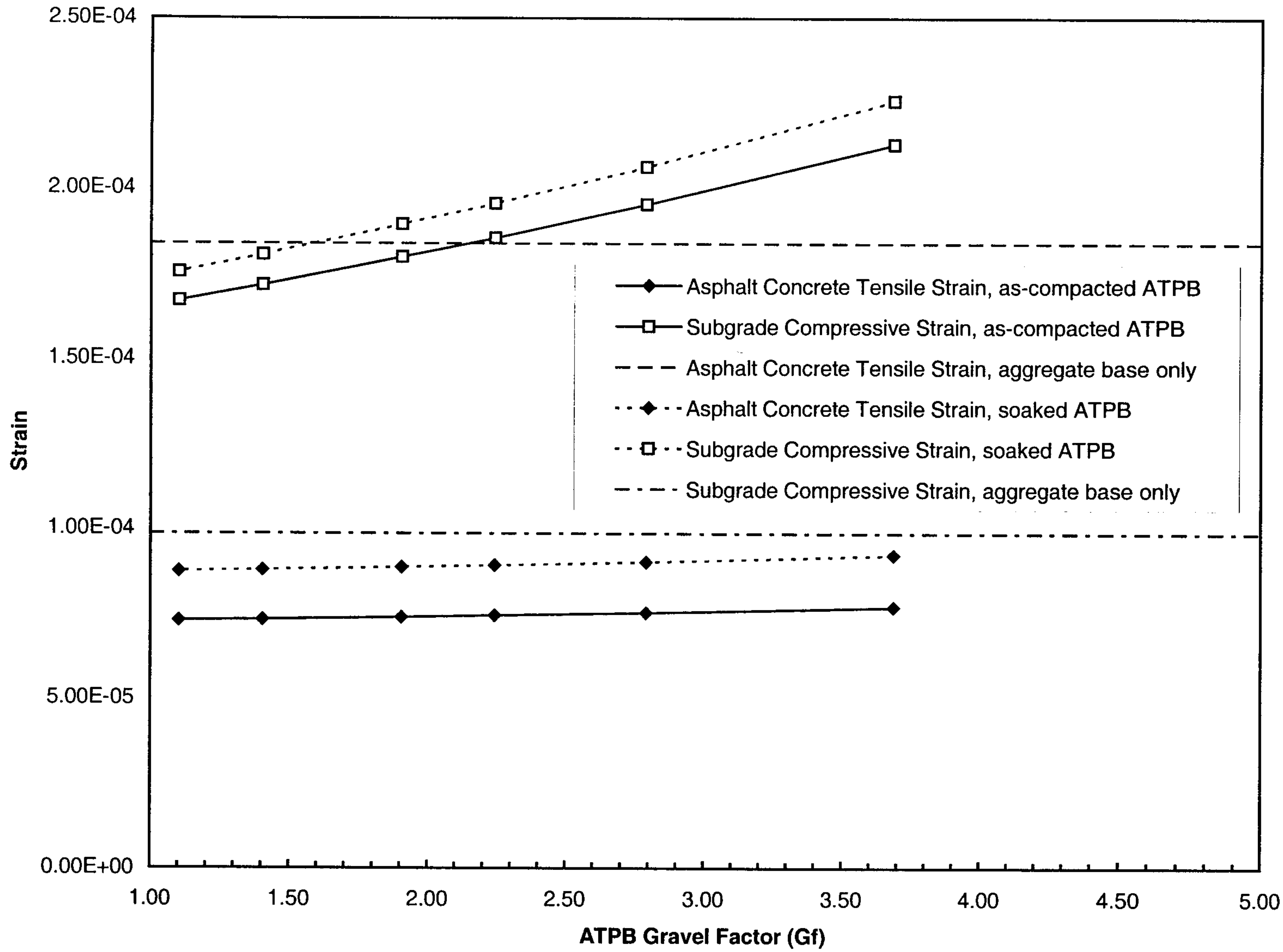

Figure 5.10. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=12, R-value=5).

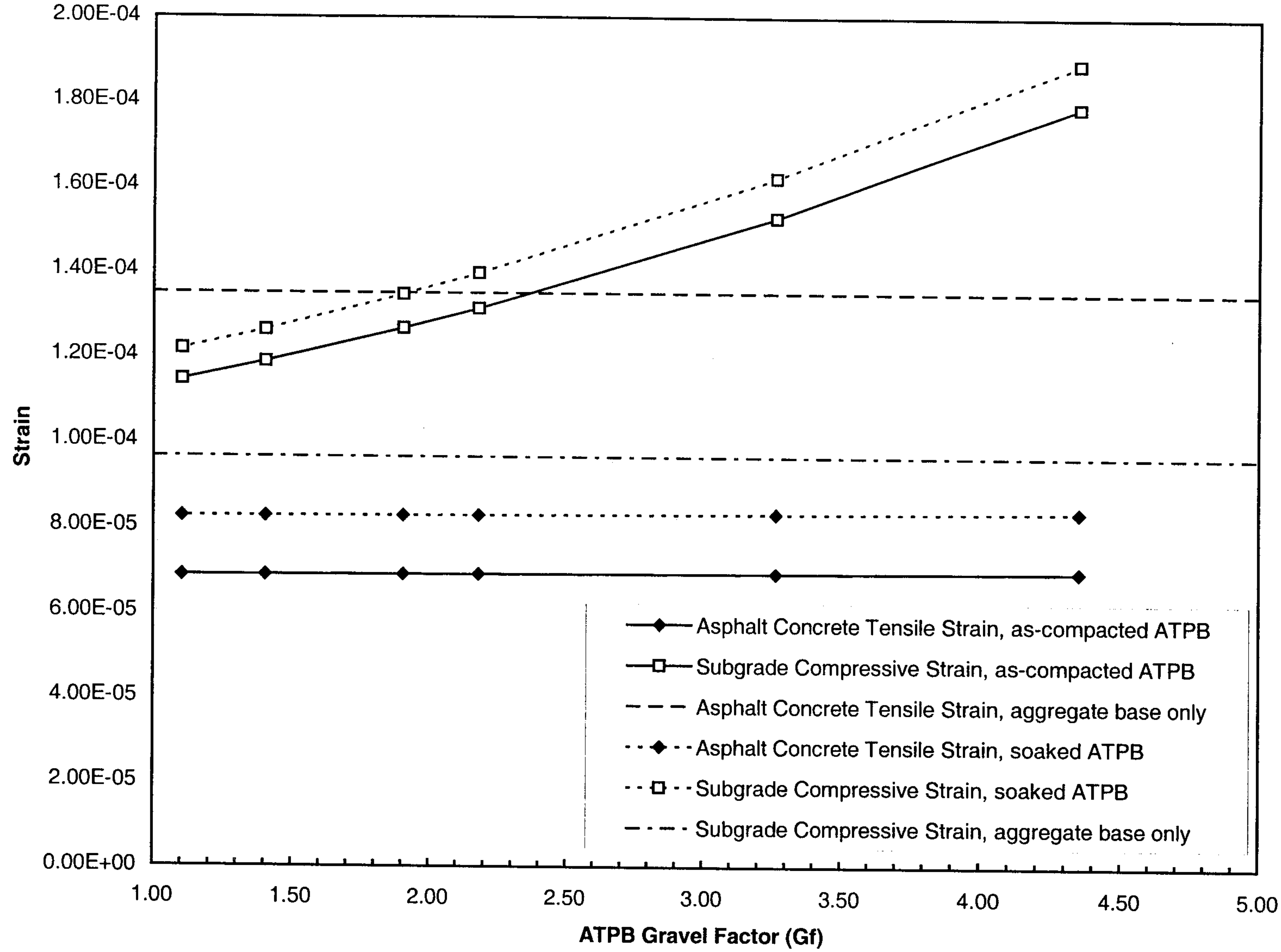

Figure 5.11. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=12, R-value=40).

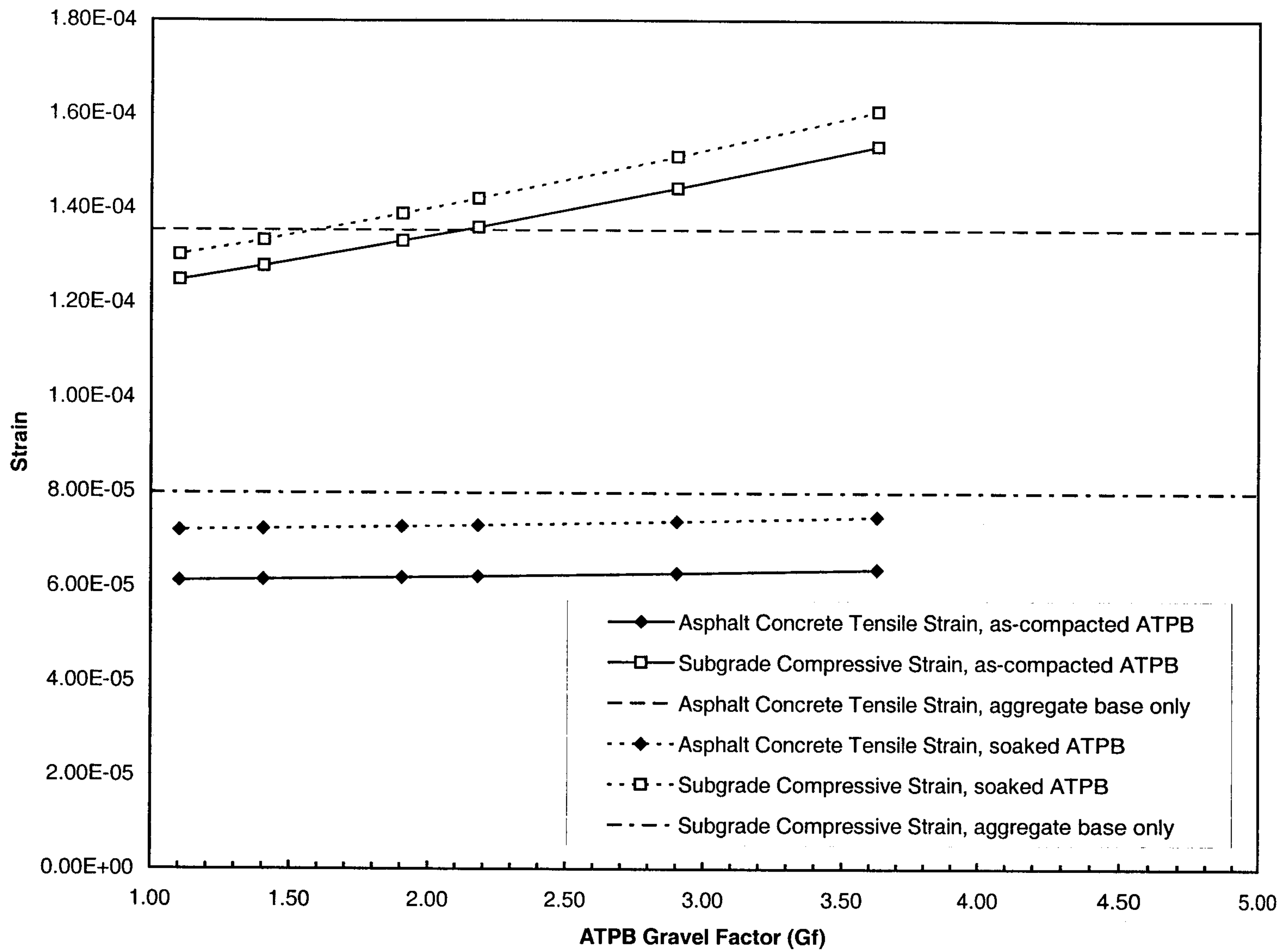

Figure 5.12. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=14, R-value=5).

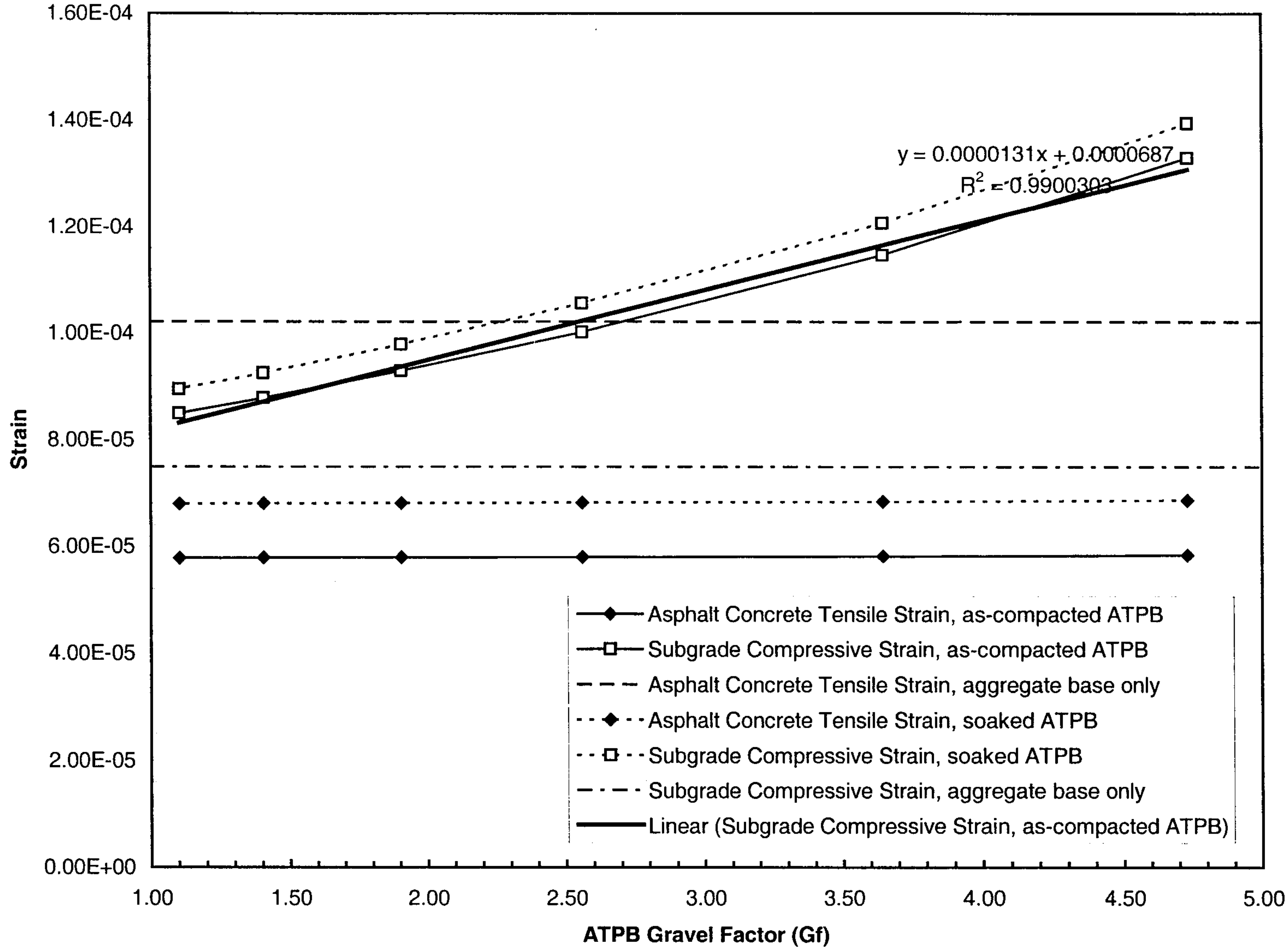

Figure 5.13. Influence of the Magnitude of the ATPB Gravel Factor on Vertical Compressive Strain at the Subgrade Surface and the Tensile Strain at the Underside of the Asphalt Concrete Layer (TI=14, R-value=40).

Table 5.9 . ATPB Gravel Factors Determined for a Range in Traffic Conditions (TI) and Subgrade Strengths (R-values).

		ATPB gravel factor estimate			
		as-compacted		soaked	
Traffic Index	Subgrade R-value	AC strain	Subgrade strain	AC strain	Subgrade strain
10	5	14.5	2.0	4.9	1.4
10	20	21.6	2.0	6.8	1.4
10	40	69.3	2.6	20.0	2.1
11	5	38.8	1.9	5.0	1.4
11	20	22.0	2.0	6.9	1.4
11	40	97.9	2.3	42.2	1.8
12	5	18.1	2.1	7.3	1.6
12	20	26.4	2.1	10.2	1.5
12	40	101.3	2.3	44.0	1.9
13	5	26.4	2.1	8.2	1.7
13	20	48.9	2.0	10.5	1.5
13	40	-39.0	2.6	38.1	2.1
14	5	20.9	2.1	8.4	1.6
14	20	30.6	2.0	11.8	1.5
14	40	106.7	2.5	38.5	2.2
		Average	2.2	Average	1.7

Table 5.10 Values of the ATPB Gravel Factor Based on Subgrade Strain for Both the As-Compacted and Soaked Conditions as a Function of R-value and TI.

SG R-value	As-Compacted	Soaked
5	2.0	1.5
20	2.0	1.5
40	2.4	2.0
Traffic Index	As-Compacted	Soaked
10	2.2	1.6
11	2.1	1.6
12	2.1	1.6
13	2.3	1.8
14	2.2	1.7

The values of $G_{f_{ATPB}}$ as a function of TI and R-value for both the as-compacted and soaked conditions are summarized in Table 5.10. It will noted that when one subgrade is strong the values for $G_{f_{ATPB}}$ are slightly higher than for the weaker subgrade conditions. From this information, it would seem reasonable to use a $G_{f_{ATPB}}$ for the ATPB in the range 2.0 to 2.4 if the material is not subject to stripping. If there is more water damage, then a lesser value (e.g., approximately 1.7) would appear reasonable.

5.6 Findings

The results of the analysis presented in this chapter suggest that ATPB that is not water damaged can add considerable structural capacity to asphalt concrete pavements (ACP). The results indicate that inclusion of ATPB material may substantially improve the resistance of Caltrans pavements to fatigue cracking compared to pavements containing aggregate base alone, provided the ATPB is not susceptible to damage from exposure to water.

The analyses also indicate that the gravel factor when the ATPB is not susceptible to water damage should be higher than 1.4: in the range 2.0 to 2.4 depending on subgrade R-value. If water damage does take place, a lower subgrade R-value in the range 1.7 to the current value of 1.4 is warranted. These values, as stated earlier, are associated with considerations of subgrade strain.

While the ATPB does reduce the strain in the asphalt concrete as well as in the surface of the subgrade, it is likely this role of reducing both strain levels could be more reliably achieved by an equivalent thickness of asphalt concrete (76 mm or less) which has considerably greater stiffness and water resistance than does ATPB as currently designed and constructed. Moreover, further improvement in fatigue resistance (and water resistance) could be accomplished by the use of a "rich-bottom" design as illustrated in Reference *(18)*.

In general, these results suggest that Caltrans should rethink its policies for ATPB both with respect to its use and mix design considerations. Recommendations in this regard are included in Chapter 6.

6.0 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

This report is one of a series detailing the results of the CAL/APT program being performed jointly by UCB and Caltrans. It contains an evaluation of the performance of asphalt treated permeable base (ATPB) in asphalt concrete pavements based on observation, laboratory testing of ATPB, and computer simulation of representative pavement structures. The evaluation includes: 1) a summary of Caltrans experience with ATPB and drainage systems, including their development and implementation, and observations of field performance with respect to stripping and maintainability; 2) a summary of the characteristics and performance of ATPB materials and drainage systems used by two other highway agencies; 3) results of a laboratory investigation of the stiffness and permanent deformation characteristics of ATPB mixes, including the effects of soaking and loading while saturated on these parameters; and, 4) the results of analyses of representative pavement structures to quantify the expected effects of as-compacted and soaked ATPB on pavement performance.

The objectives of the studies reported herein were to:

1. obtain, through laboratory testing, an indication of the effects of water on the stiffness and permanent deformation characteristics of ATPB;
2. relate the soaking performed in the laboratory, including its effects on stiffness and permanent deformation, to field conditions;
3. understand the structural effects on pavement performance of pavement structures with ATPB, including the effects of soaking the ATPB (this phase is intended to

provide a bridge between HVS test results on pavements containing ATPB in a relatively dry state and field performance of pavements in which the ATPB will likely be subjected to soaking);

4. understand the design philosophy that has led to the use of ATPB in Caltrans pavements and review the implementation of that philosophy to date; and
5. evaluate this information in order to make recommendations for the use of ATPB in flexible pavements in California.

The results included in this report are intended to supplement the results of other laboratory tests, analyses, and HVS tests on drained and undrained pavement structures being performed as part of Goal 1 of the CAL/APT program Strategic Plan, which is concerned with validation existing Caltrans design procedures for both new and overlaid pavements.

The original philosophy of Caltrans was to include asphalt treated permeable materials in the pavement in order to more quickly remove both surface and subsurface water entering the pavement section, and therefore maintain the structural capacity of the unbound materials in the pavement. In practice, Caltrans has included ATPB as a part of the structural cross-section of asphalt concrete pavements for more than 15 years. During this period, some problems have been reported. Stripping of asphalt from the aggregate has been observed in some ATPB materials — this phenomenon has also been reported by other agencies using similar materials. Maintenance of edge drains has been a problem for some Caltrans districts, particularly where the drains have been added as retrofits rather than as design features in new or reconstructed pavements. In addition, several districts have reported frequent clogging of their drainage systems. These observations stress the importance of examining ATPB in a soaked state.

While the extent to which an ATPB might remain saturated has not been extensively examined, results of the study in Indiana reported herein suggest that the ATPB can remain saturated for a substantial period of time after a rain event. These results indicate that it is important to study the response of saturated ATPB and that loading representative of moving traffic be applied to the material in this condition.

Because of these observed problems and conditions, and because the HVS test on the pavement section containing the ATPB was performed with this material in the "dry" state (Section 500 RF), a series of laboratory tests were performed on ATPB. The ATPB materials tested were designed following the current Caltrans specifications and were collected during construction of the HVS pavement and a field pavement section. The laboratory-compacted specimens prepared from these materials were tested in the as-compacted state and after soaking in 20°C water for up to ten days. Permanent deformation tests using repeated loading of the triaxial specimens were also performed in the as-compacted and saturated states. The results of the laboratory tests show significant reductions in resilient modulus after soaking, increased permanent deformation rates, and loss of cohesion and stripping at particle interfaces when subjected to repeated loading while saturated.

To obtain a comparative performance in fatigue and subgrade rutting for drained (containing ATPB) and undrained (aggregate base only) pavements designed for traffic indices of 8 through 14 and subgrade R-values of 5, 20, and 40, a series of simulations were performed using the analysis procedure reported in Reference. *(19)* The pavement layer thicknesses were designed using the Caltrans pavement design procedure, NEWCON 90. Performance of the drained pavements was simulated using ATPB stiffnesses obtained from the laboratory tests on the ATPB in the as-compacted state and after ten days of soaking. The results of the simulations indicated that the drained pavements would have substantially longer service lives than would the undrained pavements. Performance of the drained pavements containing ATPB that had been soaked for ten days was poorer than that of pavements in which the ATPB was in the as-compacted state, however, the performance of the drained pavements in both conditions still exceeded the performance of the undrained pavements.

6.2 Conclusions

From the results presented in this report, the following conclusions appear warranted:

1. Although it was not the original intent, it appears that in addition to any benefits provided by improved drainage of the pavement, inclusion of an ATPB layer beneath the asphalt concrete layer can increase the structural capacity of Caltrans asphalt-concrete pavements with respect to fatigue cracking and subgrade rutting, provided that the ATPB is resistant to stripping, loss of cohesion, and stiffness reduction from water damage. For these conditions, the ATPB gravel factor can be increased to a value on the order of 2.0 from its current value of 1.4. If, on the other hand, some water damage occurs, leading to reduction in stiffness, a gravel factor in the range 1.7 to 1.4 can be considered depending on the degree of water damage anticipated.
2. ATPB materials may be susceptible to stripping, loss of cohesion, and stiffness reduction from prolonged exposure to water as they are currently designed and constructed by Caltrans.

3. It is likely that the specifications for ATPB can be adjusted to substantially decrease the probability and extent of water damage, while maintaining permeability characteristics similar to those of the current material.

4. Water damage to ATPB and other pavement layers can also be decreased by any combination of the following additional actions:

    a. ensure that edge and transverse drains for the ATPB layer are adequately designed to prevent prolonged entrapment of water in the ATPB,

    b. ensure that edge and transverse drains for the ATPB layer are adequately maintained to prevent prolonged entrapment of water in the ATPB, or design them so that maintenance requirements are within budget and personnel constraints,

    c. decrease the flow of surface water through the asphalt concrete surface by decreasing its permeability through more stringent compaction requirements for the asphalt concrete than currently specified.

5. To reduce the potential for clogging of ATPB by fines, the use of a filter layer, either a geotextile or designed soil filter, should be evaluated.

6.3 Discussion and Recommendations

6.3.1 Discussion

From the information presented in this report, it would appear worthwhile for Caltrans to reconsider its policy on the use of ATPB in pavement sections. In this regard, the location of the

ATPB will likely influence any decision. For example, if seepage is occurring into the pavement section from the subgrade, then the use of the ATPB is warranted. However, for this application, requirements for the ATPB should be modified. In addition, a suitable filter layer based on reliable filter design principles should be incorporated.

Current practice in which the ATPB is placed directly under the asphalt concrete layer should be reconsidered. The argument for placing ATPB in this location is that it can intercept water that enters through the pavement surface. The two major reasons that water may enter the pavement structure through the pavement surface are the existence of cracks in the surface and/or a permeable asphalt concrete layer. The need for ATPB in this location could be eliminated by reducing the permeability of the asphalt concrete, which can be achieved through mix design, improved compaction, and construction practices, and by mitigating the potential for load associated cracking through improved compaction and incorporation of sufficient asphalt concrete thicknesses. Asphalt concrete thicknesses sufficient to reduce the probability of fatigue cracking can be designed using analytically-based methodology of the type discussed herein.

In dense-graded asphalt concrete, increased compaction can contribute to a reduction in the permeability of asphalt concrete for air-void contents greater than about 6 to 8 percent, by decreasing the interconnection between voids. Compaction to air-voids contents below about 6 to 8 percent has little effect on permeability because voids are no longer connected and only the size and quantity of air voids are reduced. *(24)* Construction practices, such as roller type and roller speed, are also important in creating interconnection of air-voids, and therefore influence permeability. *(25)*

The value of the ATPB as a structural layer when it has no role as a drainage layer does not justify its use. Data reported herein suggest s stiffness values on the order of 1×10^6 kPa (150,000 psi) for ATPB material in the dry state at 20°C. At the same temperature, conventional asphalt concrete will have stiffness values in the range 5.5 to 7×10^6 kPa (800,000 to 1,000,000 psi). If the ATPB is saturated and the material is sensitive to water, the stiffness may be reduced to a value of about 5×10^5 kPa (75,00 psi), about one-half that in the dry state. While the stiffness values in both the dry and wet state appear larger than representative stiffness values for untreated granular bases, improved pavement performance could, from a cost standpoint, be better achieved by proper mix design and thickness selection (e.g. use of the "rich bottom" design) and through improved construction practices, particularly asphalt concrete compaction. *(17,18)*

If a decision is made to continue the use of ATPB directly beneath the asphalt concrete layer, then the actions recommended above for the use of ATPB in pavements with subsurface seepage should be undertaken for its use beneath the asphalt concrete layer as well.

6.3.2 Recommendations

Based on the information contained herein, the following recommendations are presented:

1. ***Eliminate ATPB as a drainage layer directly beneath the asphalt concrete layer in the structural pavement section.*** Adoption of this recommendation will require that Caltrans improve construction specifications for asphalt concrete by requiring a higher degree of compaction in this material. In addition, it requires that the crushed

stone base be properly compacted like the asphalt concrete, to a higher degree of compaction as compared to present standards, probably in the range of 100 to 105 percent of the equivalent of AASHTO T180. This level of compaction will decrease the permeability of the crushed stone base, and make its strength and stiffness characteristics less susceptible to increases in water content.

Associated with Recommendation (1) is Recommendation (2):

2. ***Use the analytically-based design procedure, which has been calibrated with the HVS tests performed under Goal 1 of the CAL/APT program, to determine the thickness of the asphalt concrete layer and incorporating the concept of "rich-bottom" design.*** Adoption of this recommendation will likely provide a higher level of reliability relative to fatigue cracking and, in conjunction with Recommendation (1), reduce the permeability of the asphalt concrete layer to surface water.

3. ***Develop requirements to improve water damage resistance of ATPB Materials.*** The resistance of ATPB materials to water damage (stripping, loss of cohesion, and loss of stiffness) can be significantly reduced by changes in the specifications for ATPB. The most likely changes are increased asphalt content and changes in binder specification including the use of asphalt-rubber. Any changes in the ATPB specification would need to ensure sufficient permeability and constructability comparable to materials currently in use. Recommendation (3) will require a laboratory test program using triaxial testing, including development of a procedure for loading the specimen while in a saturated state. Inclusion of flow of water through the material in the conditioning program should be considered.

4. ***Define methods to maintain the drainage capacity of ATPB layers.*** If ATPBs are to remain effective as drainage layers, it will be necessary to insure that:

    a. adequate filter layers are provided adjacent to the ATPB to minimize the intrusion of fines; and

    b. edge and transverse drains are maintained to prevent their filling with fines or becoming clogged.

    The current practice of using a heavy prime coat on the aggregate base as the filter material should be evaluated to ascertain its effectiveness. Guidelines should be developed for proper design of filters using either soils or geotextiles. Recommended maintenance practices for edge and transverse drains should be established and distributed to the Districts.

5. ***If the recommendations are followed to improve the resistance of ATPB to the action of water, then its gravel factor should be increased to a value of 2.0.***

6. ***HVS tests should be performed on the pavements used in the Goal 1 Study when subjected to surface and subsurface water to provide validation of these recommendations.*** The sections would be trafficked by the HVS while in a wet condition; the tests should provide results regarding:

    1. Validation of analyses of drained and undrained pavement fatigue and subgrade rutting performance when subjected to water.

    2. Quantitative measurement of the effectiveness of the ATPB in removing water from the drained pavements compared to the undrained pavements (water could be introduced from surface locations).

3. Measurement of the effects of water on the stiffness and rutting performance of each layer in the pavements and overall pavement strength under wet conditions as compared to dry conditions.

4. Validation for laboratory test data relating the effects of water content or degree of saturation on subgrade stiffness. This work was largely completed as part of the initial work on Goal 1 of the CAL/APT Strategic Plan. *(10)*

In addition to the recommendations listed above which are directly related to Goal 1, other investigations could include:

1. Measurement of water infiltration rates through a cracked pavement surface and through the base and subbase layers when subjected to trafficking. As a part of this study the following could be incorporated:

    a. Application of several maintenance measures to reduce surface infiltration and measurement of their effectiveness under trafficking and wet conditions. Example treatments could include crack sealing, chip seal, slurry seal and/or rubberized chip seal.

    b. Blocking of the edge drain to measure the effect of edge drain maintenance on pavement performance for drained and undrained pavements.

2. Develop validation data for ground penetrating radar (GPR), electrical resistance tomography, and other advanced methods of water content measurements below the pavement surface.

These latter two recommendations could be developed as part of a new goal in the CAL/APT strategic plan.

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APPENDIX A: EFFECTS OF FREQUENCY AND LOAD OF VIBRATORY COMPACTOR USED FOR PREPARATION OF ATPB LABORATORY SPECIMENS

The purpose of this experiment was to define the frequency and load applied to the ATPB material by the air hammer used for compaction. The equipment setup for this evaluation is shown in Figure A-1. A split mold was seated and bolted on a circular mold support and a load cell was located in the middle of mold support. Two tie rods were connected to the top and bottom of the load cell and thus formed a unit to bridge the bottom plate and bottom block underneath the ATPB material.

The data was sampled at the rate of 200 points per 5 seconds. Time and applied compaction load were recorded automatically by the data collection system. Total duration of this experiment was 60 seconds, corresponding to the compaction time for each lift. Figure A-2 illustrates the time series plot of applied compaction load versus time. This figure was obtained under the "normal operation" of the air vibratory hammer. That is, the hammer operator made no extra effort to push down the hammer or perform any other operation.

Measurement of compaction frequency was determined by counting the number of observed peaks during 60 seconds; this corresponded to a frequency of about 12.5 Hz. The average applied compaction load, which represented the average of all the collected data, was about 160 N. For a 100-mm (4-in.) diameter specimen, this would correspond to a pressure of 5.1 kPa. However, in Figure A-2, it can be observed that the peak values of compaction load ranged from 200 N to 650 N, corresponding to pressures of 6.4 kPa to 20.7 kPa (0.9 psi to 3.0 psi) for a 100-mm (4-in.) diameter specimen.

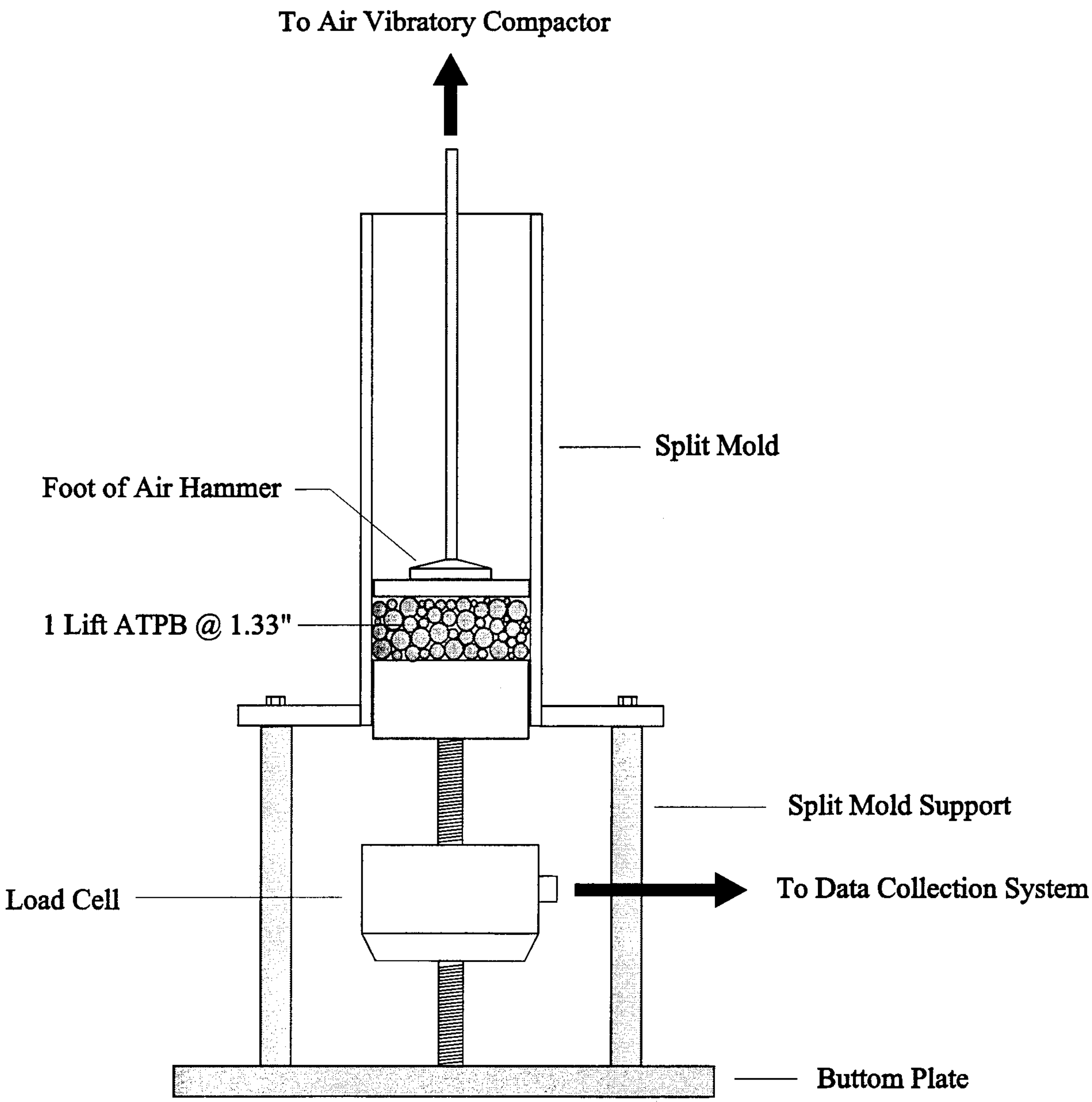

Figure A1. Frequency measurement setup for air vibratory compaction.

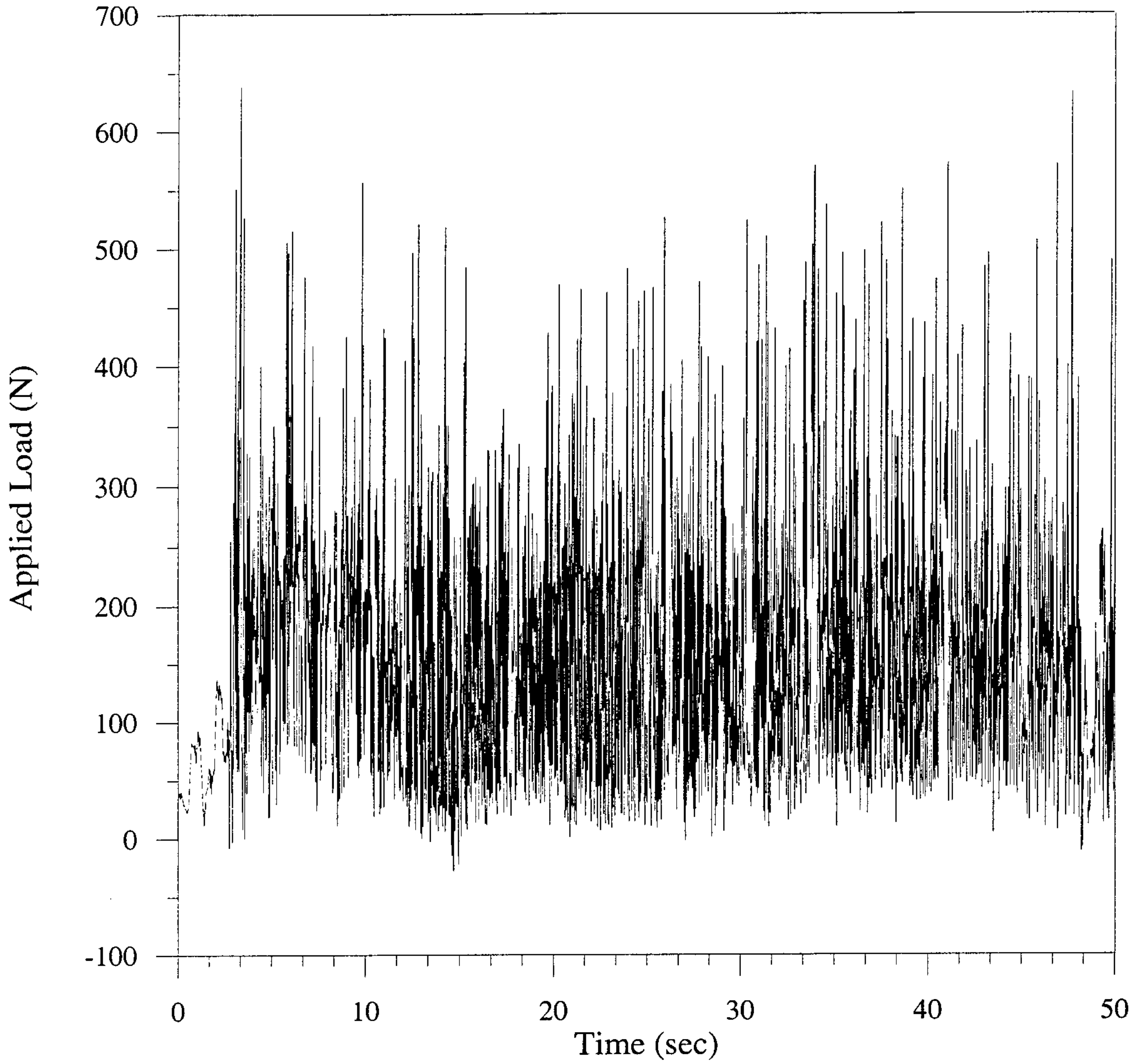

Figure A2. The time series plot of applied vibratory compaction load.

APPENDIX B: THE RESILIENT MODULUS TEST RESULTS OF THE CAL/APT GOAL 1 SPECIMENS

Table B1 The Resilient Modulus Test Results of ATPB Specimen 1 with AV=35.7% (CAL/APT Goal 1).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' soak)			RM10 (after 7 more days' soak)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	92	299	970280	80	287	970280	85	292	557971
69	138	135	342	923162	135	342	923162	142	349	539924
69	207	211	418	882385	207	414	882385	212	419	553181
103.5	110.4	116	426.5	966856	106	416.5	966856	114	424.5	643825
103.5	207	207	517.5	900909	199	509.5	900909	213	523.5	628835
103.5	310.5	311	621.5	872875	295	605.5	872875	319	629.5	624320
138	138	141	555	982090	149	563	982090	142	556	685697
138	276	282	696	898225	328	742	898225	281	695	723798
138	414	418	832	855953	411	825	855953	426	840	720411
207	207	213	834	782455	213	834	782455	211	832	789536
207	414	426	1047	692257	411	1032	692257	422	1043	779762
138	276	282	696	727468	290	704	727468	281	695	608220
69	138	144	351	814272	147	354	814272	143	350	505875
34.5	82.8	85	188.5	819266	87	190.5	819266	85	188.5	422892
34.5	138	139	242.5	762370	142	245.5	762370	139	242.5	444796
34.5	82.8	85	188.5	867283	86	189.5	867283	85	188.5	437257
69	138	137	344	842504	139	346	842504	142	349	545800
138	276	281	695	918404	278	692	918404	283	697	727205
207	414	422	1043	892820	416	1037	892820	425	1046	814837
207	207	212	833	890758	211	832	890758	214	835	802829
138	414	424	838	771367	417	831	771367	427	841	701623
138	276	278	692	756042	279	693	756042	279	693	640255
138	138	142	556	746371	139	553	746371	145	559	659230
103.5	310.5	315	625.5	747500	328	638.5	747500	319	629.5	606892
103.5	207				215	525.5	495388	214	524.5	578855
103.5	110.4	115	425.5	808456	111	421.5	808456	113	423.5	585110
69	207	215	422	731198	215	422	731198	211	418	519925
69	138	145	352	769201	142	349	769201	141	348	520128
69	82.8	87	294	817800	82	289	817800	86	293	524119

Table B2 **The Resilient Modulus Test Results of ATPB Specimen 2 with AV=34.5% (CAL/APT Goal 1).**

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' saturated PD test)			RM10 (after 7 more days' saturated PD test)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	89	296	1692236	82	289	1426676	(specimen failed during PD test)		
69	138	140	347	1553684	138	345	1065618			
69	207	213	420	1444742	208	415	1036691			
103.5	110.4	116	426.5	1598734	115	425.5	1317774			
103.5	207	211	521.5	1416220	207	517.5	1163798			
103.5	310.5	316	626.5	1317254	313	623.5	1086752			
138	138	142	556	1572408	140	554	1415334			
138	276	287	701	1365812	281	695	1171286			
138	414	425	839	1284910	417	831	1111868			
207	207	214	835	1503992	210	831	1330592			
207	414	425	1046	1327306	423	1044	1188806			
138	276	285	699	1200386	286	700	1057472			
69	138	145	352	1499154	141	348	956956			
34.5	82.8	86	189.5	1380560	84	187.5	1091897			
34.5	138	144	247.5	1395436	138	241.5	909935			
34.5	82.8	90	193.5	1469902	85	188.5	1052254			
69	138	140	347	1411876	138	345	1048602			
138	276	279	693	1466084	280	694	1098318			
207	414	420	1041	1382750	419	1040	1293426			
207	207	211	832	1348826	210	831	1423266			
138	414	420	834	1259722	420	834	1103068			
138	276	284	698	1283132	281	695	1162796			
138	138	142	556	1304016	141	555	1285834			
103.5	310.5	321	631.5	1293650	315	625.5	971324			
103.5	207	215	525.5	1428500	212	522.5	1059308			
103.5	110.4	114	424.5	1795124	113	423.5	1217156			
69	207	213	420	1441708	214	421	961410			
69	138	147	354	1447450	144	351	1034859			
69	82.8	88	295	1514842	91	298	1198686			

Table B3 The Resilient Modulus Test Results of ATPB Specimen 3 with AV=34.2% (CAL/APT Goal 1).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' dry PD test)			RM10 (after 7 more days' dry PD test)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	90	297	2334640	87	294	1972400	85	292	677517
69	138	140	347	2295808	136	343	1887320	142	349	770157
69	207	214	421	1963990	214	421	1660208	208	415	1254092
103.5	110.4	116	426.5	2155620	111	421.5	2288028	115	425.5	1289362
103.5	207	214	524.5	1958566	206	516.5	1762758	218	528.5	1296574
103.5	310.5	316	626.5	1885202	320	630.5	1501232	328	638.5	1186594
138	138	146	560	2567804	137	551	2300854	147	561	1150847
138	276	281	695	1988298	279	693	1627470	287	701	1250108
138	414	421	835	1858402	418	832	940097	416	830	1249234
207	207	212	833	1546120	208	829	1133304	207	828	1514646
207	414	426	1047	1586536	416	1037	795730	432	1053	1218444
138	276	281	695	1863296	274	688	1135692	287	701	1243632
69	138	145	352	1953832	138	345	1762304	146	353	1261174
34.5	82.8	90	193.5	2472886	81	184.5	1710696	86	189.5	1305174
34.5	138	144	247.5	1953502	142	245.5	1617140	145	248.5	1255806
34.5	82.8	88	191.5	2368038	86	189.5	2056110	89	192.5	1328588
69	138	141	348	1950802	140	347	1995642	139	346	1357644
138	276	286	700	1837634	279	693	1472324	284	698	1396854
207	414	423	1044	1591632	420	1041	849912	430	1051	1324162
207	207	218	839	1814508	208	829	1171338	219	840	1706690
138	414	424	838	1794518	421	835	1114242	428	842	1253994
138	276	283	697	1802270	281	695	1129482	285	699	1351962
138	138	145	559	1480134	140	554	2462974	140	554	1496166
103.5	310.5	323	633.5	1731806	318	628.5	1475198	318	628.5	1203164
103.5	207	215	525.5	1815696	214	524.5	1655188	210	520.5	1291400
103.5	110.4	114	424.5	1552920	115	425.5	2028028	112	422.5	1518152
69	207	211	418	1722492	206	413	1656550	211	418	1436810
69	138	147	354	1877656	141	348	1693030	140	347	1484752
69	82.8	84	291	2097360	85	292	1814482	84	291	1415268

Table B4 The Resilient Modulus Test Results of ATPB Specimen 4 with AV=35.0% (CAL/APT Goal 1).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' saturated PD test)			RM10 (after 7 more days' saturated PD test)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	86	293	1029018	87	294	564025	(specimen failed during PD test)		
69	138	141	348	1069667	142	349	444402			
69	207	217	424	994763	228	435	454401			
103.5	110.4	114	424.5	1171672	118	428.5	568213			
103.5	207	210	520.5	1033732	211	521.5	532510			
103.5	310.5	318	628.5	988330	314	624.5	528948			
138	138	144	558	1180132	140	554	619208			
138	276	281	695	1066176	280	694	553953			
138	414	426	840	975506	425	839	557878			
207	207	217	838	1202652	207	828	679995			
207	414	423	1044	1066338	424	1045	634473			
138	276	284	698	995784	282	696	565003			
69	138	143	350	1006981	142	349	472455			
34.5	82.8	85	188.5	1039011	93	196.5	518184			
34.5	138	140	243.5	898101	152	255.5	481772			
34.5	82.8	92	195.5	1095528	88	191.5	430534			
69	138	145	352	1035882	141	348	475894			
138	276	276	690	1104202	279	693	595086			
207	414	422	1043	1135864	419	1040	682129			
207	207	211	832	1195184	207	828	680279			
138	414	422	836	1009008	423	837	586055			
138	276	279	693	1030842	282	696	566164			
138	138	140	554	1133664	141	555	564170			
103.5	310.5	319	629.5	971980	318	628.5	522679			
103.5	207	210	520.5	1025343	213	523.5	495388			
103.5	110.4	113	423.5	1257904	113	423.5	501720			
69	207	215	422	992369	216	423	446450			
69	138	143	350	1027215	140	347	416216			
69	82.8	89	296	1246944	86	293	429932			

Table B5 The As-Compacted Resilient Modulus of Four Specimens (CAL/APT Goal 1).

loading sequence		Sequence 1 (as-compacted RM)		Sequence 2 (as-compacted RM)		Sequence 3 (as-compacted RM)		Sequence 4 (as-compacted RM)	
confining pressure (kPa)	deviator stress (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)
69	82.8	299	970280	296	1692236	297	2334640	293	1029018
69	138	342	923162	347	1553684	347	2295808	348	1069667
69	207	418	882385	420	1444742	421	1963990	424	994763
103.5	110.4	426.5	966856	426.5	1598734	426.5	2155620	424.5	1171672
103.5	207	517.5	900909	521.5	1416220	524.5	1958566	520.5	1033732
103.5	310.5	621.5	872875	626.5	1317254	626.5	1885202	628.5	988330
138	138	555	982090	556	1572408	560	2567804	558	1180132
138	276	696	898225	701	1365812	695	1988298	695	1066176
138	414	832	855953	839	1284910	835	1858402	840	975506
207	207	834	782455	835	1503992	833	1546120	838	1202652
207	414	1047	692257	1046	1327306	1047	1586536	1044	1066338
138	276	696	727468	699	1200386	695	1863296	698	995784
69	138	351	814272	352	1499154	352	1953832	350	1006981
34.5	82.8	188.5	819266	189.5	1380560	193.5	2472886	188.5	1039011
34.5	138	242.5	762370	247.5	1395436	247.5	1953502	243.5	898101
34.5	82.8	188.5	867283	193.5	1469902	191.5	2368038	195.5	1095528
69	138	344	842504	347	1411876	348	1950802	352	1035882
138	276	695	918404	693	1466084	700	1837634	690	1104202
207	414	1043	892820	1041	1382750	1044	1591632	1043	1135864
207	207	833	890758	832	1348826	839	1814508	832	1195184
138	414	838	771367	834	1259722	838	1794518	836	1009008
138	276	692	756042	698	1283132	697	1802270	693	1030842
138	138	556	746371	556	1304016	559	1480134	554	1133664
103.5	310.5	625.5	747500	631.5	1293650	633.5	1731806	629.5	971980
103.5	207			525.5	1428500	525.5	1815696	520.5	1025343
103.5	110.4	425.5	808456	424.5	1795124	424.5	1552920	423.5	1257904
69	207	422	731198	420	1441708	418	1722492	422	992369
69	138	352	769201	354	1447450	354	1877656	350	1027215
69	82.8	294	817800	295	1514842	291	2097360	296	1246944

APPENDIX C: THE RESILIENT MODULUS TEST RESULTS OF THE VASCO ROAD SPECIMENS

Table C1 The Resilient Modulus Test Results of Specimen CC5R of the Contra Costa County Vasco Road Project (AV = 32.1%, Nominal AC = 2.0%).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' soak)			RM10 (after 7 more days' soak)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	81.5	288.5	1900188	82.1	289.1	917475	81.8	288.8	1108250
69	138	139.7	346.7	1435748	141	348	908465	141.3	348.3	1153968
69	207	213.8	420.8	1248798	212.5	419.5	962765	210.4	417.4	1258058
103.5	110.4	112.3	422.8	1716020	112	422.5	1001898	109.6	420.1	1055830
103.5	207	210.5	521	1307124	211.5	522	1062972	208.1	518.6	1413008
103.5	310.5	319.1	629.6	1179962	314.9	625.4	1040754	316.6	627.1	1260788
138	138	138.1	552.1	1648394	142.7	556.7	1068810	137.1	551.1	1134722
138	276	284.1	698.1	1261088	279.6	693.6	1122734	283.3	697.3	1384520
138	414	427.2	841.2	1081636	421.2	835.2	1025220	420.4	834.4	1224960
207	207	206.9	827.9	1466468	212.3	833.3	1270418	210.9	831.9	1433596
207	414	422.9	1043.9	1186468	418.5	1039.5	1109456	419.7	1040.7	1324788
138	276	280.7	694.7	1193780	280	694	997557	277.5	691.5	1243878
69	138	135.9	342.9	1330226	135.2	342.2	898404	137.1	344.1	1089156
34.5	82.8	80.8	184.3	1744282	81.6	185.1	843453	83.6	187.1	872016
34.5	138	141.1	244.6	1254570	141.2	244.7	796928	138.7	242.2	1048199
34.5	82.8	83.2	186.7	1711504	81.7	185.2	818981	81.7	185.2	890876
69	138	139.3	346.3	1338160	140.5	347.5	904446	140.3	347.3	1147682
138	276	280.3	694.3	1235194	279.8	693.8	1086720	280.2	694.2	1275114
207	414	427.2	1048.2	1209898	419.4	1040.4	1107024	422.1	1043.1	1290568
207	207	209.3	830.3	1383486	208.3	829.3	1187596	212.5	833.5	1338588
138	414	424.7	838.7	1070310	421.5	835.5	989913	420.8	834.8	1143986
138	276	279.8	693.8	1140522	283.5	697.5	975125	280.8	694.8	1213884
138	138	138.3	552.3	1399884	141.2	555.2	1007614	139.1	553.1	1273626
103.5	310.5	314.3	624.8	1069066	317.3	627.8	930621	314.2	624.7	1111734
103.5	207	207.7	518.2	1124246	210.3	520.8	924968	209.5	520	1205792
103.5	110.4	107.2	417.7	1416158	110.6	421.1	932900	108.8	419.3	1104636
69	207	209.8	416.8	1078980	211.1	418.1	859224	212.6	419.6	1098032
69	138	135.1	342.1	1204514	138.3	345.3	863298	139.5	346.5	1094834
69	82.8	81	288	1623222	81.6	288.6	888697	80.9	287.9	909092

Table C2 The Resilient Modulus Test Results of Specimen CC1L of the Contra Costa County Vasco Road Project (AV = 34.8%, Nominal AC = 2.5%).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' soak)			RM10 (after 7 more days' soak)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	80	287	2255300	82.7	289.7	1467026	97.2	304.2	2225580
69	138	139.3	346.3	2043374	143.2	350.2	2019084	141	348	1790058
69	207	208.8	415.8	1695216	207	414	1832948	209.4	416.4	1799216
103.5	110.4	120	430.5	2084584	106	416.5	2163828	116.1	426.6	2189826
103.5	207	217.3	527.8	1835506	210.7	521.2	1896390	207.8	518.3	1926698
103.5	310.5	317.7	628.2	1605710	315.4	625.9	1793634	314.7	625.2	1685414
138	138	139	553	1739570	146.6	560.6	2379056	145.1	559.1	2112938
138	276	279.7	693.7	1720044	278.8	692.8	1963048	278.5	692.5	1827522
138	414	419.5	833.5	1566560	424.2	838.2	1666084	420.8	834.8	1585138
207	207	208.3	829.3	1985040	211.5	832.5	2207274	209.6	830.6	2311224
207	414	420.9	1041.9	1721912	415.9	1036.9	1843282	418.7	1039.7	1686420
138	276	281.4	695.4	1697024	282.2	696.2	1871970	289.8	703.8	1764190
69	138	139.8	346.8	1874954	141.2	348.2	2053540	145.8	352.8	1700094
34.5	82.8	88.4	191.9	1869602	87.5	191	1832924	87.8	191.3	1839246
34.5	138	144.2	247.7	1697710	152.6	256.1	1983774	149.1	252.6	1563826
34.5	82.8	86.9	190.4	1788488	83.2	186.7	1872264	86.2	189.7	1597652
69	138	141	348	1888244	151.1	358.1	2196380	143.4	350.4	1748118
138	276	280.9	694.9	1800682	276.4	690.4	2096292	279.8	693.8	1722304
207	414	417.6	1038.6	1724282	420	1041	1860208	421.4	1042.4	1687056
207	207	209.6	830.6	1953840	211.3	832.3	2172982	210.8	831.8	1957360
138	414	420.3	834.3	1579564	423	837	1764162	422.8	836.8	1498180
138	276	280	694	1760758	274.5	688.5	1954122	281.2	695.2	1629160
138	138	142.3	556.3	2058966	147.6	561.6	2270504	142.5	556.5	1895714
103.5	310.5	315.1	625.6	1645530	311.8	622.3	1802314	332.2	642.7	1603126
103.5	207	217.9	528.4	1785518	218.1	528.6	1847896	215	525.5	1794694
103.5	110.4	114	424.5	2590764	120.2	430.7	2197326	114.3	424.8	2106370
69	207	211.6	418.6	1661772	221	428	1869324	215.9	422.9	1659244
69	138	144.5	351.5	1936552	148.1	355.1	2279378	142.5	349.5	1668366
69	82.8	85.1	292.1	1799738	84.1	291.1	1827460	86.6	293.6	1916304

Table C3 The Resilient Modulus Test Results of Specimen CC2R of the Contra Costa County Vasco Road Project (AV = 32.5%, Nominal AC = 2.0%).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' soak)			RM10 (after 7 more days' soak)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	90.6	297.6	1737260	90.8	297.8	2671054	83.7	290.7	1430980
69	138	145.1	352.1	1537934	142.8	349.8	2106230	139.7	346.7	1229642
69	207	220.1	427.1	1472844	211	418	1730884	209.7	416.7	1149996
103.5	110.4	114.9	425.4	1845806	114.3	424.8	2242964	116.3	426.8	1506924
103.5	207	212.9	523.4	1522188	212.7	523.2	1929736	198.1	508.6	1308344
103.5	310.5	316.2	626.7	1422790	315.6	626.1	1621038	329.6	640.1	1205008
138	138	143.2	557.2	1978726	140.4	554.4	2243960	141.6	555.6	1496210
138	276	282.5	696.5	1594972	283.1	697.1	1837470	281.9	695.9	1365516
138	414	423.4	837.4	1419282	424.2	838.2	1595156	418.1	832.1	1227690
207	207	211.7	832.7	1795704	210	831	2064576	207	828	1648198
207	414	423.8	1044.8	1621640	424.3	1045.3	1750636	420.4	1041.4	1328448
138	276	289.4	703.4	1588324	284.1	698.1	1762138	288	702	1350374
69	138	140.7	347.7	1520034	144	351	2073172	142.9	349.9	1222632
34.5	82.8	88.8	192.3	1653522	87.3	190.8	1985972	85.3	188.8	1272804
34.5	138	143	246.5	1391658	145.8	249.3	1833704	148.7	252.2	1218396
34.5	82.8	90.2	193.7	1680294	88.7	192.2	2016022	84.5	188	1227312
69	138	144.1	351.1	1423508	147.6	354.6	1822986	138.5	345.5	1149048
138	276	282.1	696.1	1520836	276.8	690.8	1703174	274.8	688.8	1267400
207	414	420.5	1041.5	1588952	420.8	1041.8	1761964	414.3	1035.3	1294442
207	207	209.8	830.8	1735196	209	830	2087378	205.7	826.7	1372924
138	414	424.5	838.5	1421918	418.9	832.9	1534904	419.7	833.7	1167616
138	276	285	699	1508062	277.2	691.2	1688750	280.4	694.4	1242382
138	138	140.6	554.6	1664924	138.9	552.9	1870718	139.5	553.5	1397464
103.5	310.5	321.6	632.1	1422578	315.7	626.2	1502024	311.8	622.3	1132338
103.5	207	211	521.5	1470530	209.5	520	1625964	204.6	515.1	1143356
103.5	110.4	116.6	427.1	1725302	113	423.5	2154604	119.1	429.6	1214762
69	207	212	419	1397828	211.3	418.3	1619620	205.7	412.7	1068062
69	138	144.8	351.8	1507636	141.9	348.9	1745870	145.3	352.3	1127258
69	82.8	86	293	1901072	84.7	291.7	1990208	90.7	297.7	1503280

Table C4 The Resilient Modulus Test Results of Specimen CC2L of the Contra Costa County Vasco Road Project (AV = 32.4%, Nominal AC = 2.5%).

loading sequences		RM0 (As-compacted)			RM3 (after 3 days' soak)			RM10 (after 7 more days' soak)		
confining pressure (kPa)	deviator stress (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)	actual dev. stress (kPa)	SPS (kPa)	RM (kPa)
69	82.8	90.9	297.9	964065	83.5	290.5	1718498	81.7	288.7	1537022
69	138	136.9	343.9	1035758	140	347	1591884	140.4	347.4	1420560
69	207	206.7	413.7	1548964	209.9	416.9	1597116	215.3	422.3	1408888
103.5	110.4	113.3	423.8	1124958	115.3	425.8	1817916	112.2	422.7	1770962
103.5	207	207.4	517.9	1316112	211.6	522.1	1610318	210.6	521.1	1617956
103.5	310.5	312	622.5	1388704	312.9	623.4	1534310	321.3	631.8	1337352
138	138	140.1	554.1	1184558	140.4	554.4	1800376	139.4	553.4	1955566
138	276	278.1	692.1	1370166	279.9	693.9	1632762	280	694	1448252
138	414	417.1	831.1	1580828	420.7	834.7	1522340	421.7	835.7	1286754
207	207	208.7	829.7	1472494	211.7	832.7	1788796	210	831	1765698
207	414	413.8	1034.8	1503414	418.2	1039.2	1645528	424.1	1045.1	1434056
138	276	278.8	692.8	1540452	279.5	693.5	1513840	281.9	695.9	1377764
69	138	137	344	1581618	141.5	348.5	1498110	139.3	346.3	1530166
34.5	82.8	80.2	183.7	1456140	80.9	184.4	1523668	84.7	188.2	1661802
34.5	138	136.9	240.4	1501400	140.2	243.7	1506796	139.4	242.9	1406154
34.5	82.8	82.5	186	1460982	84.2	187.7	1650114	81.8	185.3	1651614
69	138	140.3	347.3	1658072	139.9	346.9	1580124	138.3	345.3	1607504
138	276	276.3	690.3	1710182	280.7	694.7	1596210	280.4	694.4	1439182
207	414	415.2	1036.2	1630820	420.7	1041.7	1599692	422.2	1043.2	1414416
207	207	209.3	830.3	1493538	210.6	831.6	1677198	209.6	830.6	1701308
138	414	418.6	832.6	1594888	419.5	833.5	1434950	421.3	835.3	1248050
138	276	279.3	693.3	1709772	280.6	694.6	1508938	280.2	694.2	1327554
138	138	133.2	547.2	1889304	143.2	557.2	1646370	139.1	553.1	1746514
103.5	310.5	310.3	620.8	1632992	312.6	623.1	1419688	318.2	628.7	1265498
103.5	207	208.7	519.2	1664108	210.7	521.2	1427706	207.5	518	1373884
103.5	110.4	107.5	418	1708906	114.5	425	1684630	110	420.5	1646318
69	207	207.2	414.2	1630164	207.3	414.3	1361450	210.5	417.5	1297608
69	138	137.8	344.8	1790510	142.6	349.6	1479986	137.4	344.4	1574978
69	82.8	84	291	1639364	81.7	288.7	1576110	82.7	289.7	1274816

Table C5 The As-Compacted Resilient Moduli of Four Sequences of the Contra Costa County Vasco Road Project.

loading sequence		Specimen CC5R (as-compacted RM)		Specimen CC1L (as-compacted RM)		Specimen CC2R (as-compacted RM)		Specimen CC2L (as-compacted RM)	
confining pressure (kPa)	deviator stress (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)	SPS (kPa)	RM (kPa)
69	82.8	288.5	1900188	287	2255300	297.6	1737260	297.9	964065
69	138	346.7	1435748	346.3	2043374	352.1	1537934	343.9	1035758
69	207	420.8	1248798	415.8	1695216	427.1	1472844	413.7	1548964
103.5	110.4	422.8	1716020	430.5	2084584	425.4	1845806	423.8	1124958
103.5	207	521	1307124	527.8	1835506	523.4	1522188	517.9	1316112
103.5	310.5	629.6	1179962	628.2	1605710	626.7	1422790	622.5	1388704
138	138	552.1	1648394	553	1739570	557.2	1978726	554.1	1184558
138	276	698.1	1261088	693.7	1720044	696.5	1594972	692.1	1370166
138	414	841.2	1081636	833.5	1566560	837.4	1419282	831.1	1580828
207	207	827.9	1466468	829.3	1985040	832.7	1795704	829.7	1472494
207	414	1043.9	1186468	1041.9	1721912	1044.8	1621640	1034.8	1503414
138	276	694.7	1193780	695.4	1697024	703.4	1588324	692.8	1540452
69	138	342.9	1330226	346.8	1874954	347.7	1520034	344	1581618
34.5	82.8	184.3	1744282	191.9	1869602	192.3	1653522	183.7	1456140
34.5	138	244.6	1254570	247.7	1697710	246.5	1391658	240.4	1501400
34.5	82.8	186.7	1711504	190.4	1788488	193.7	1680294	186	1460982
69	138	346.3	1338160	348	1888244	351.1	1423508	347.3	1658072
138	276	694.3	1235194	694.9	1800682	696.1	1520836	690.3	1710182
207	414	1048.2	1209898	1038.6	1724282	1041.5	1588952	1036.2	1630820
207	207	830.3	1383486	830.6	1953840	830.8	1735196	830.3	1493538
138	414	838.7	1070310	834.3	1579564	838.5	1421918	832.6	1594888
138	276	693.8	1140522	694	1760758	699	1508062	693.3	1709772
138	138	552.3	1399884	556.3	2058966	554.6	1664924	547.2	1889304
103.5	310.5	624.8	1069066	625.6	1645530	632.1	1422578	620.8	1632992
103.5	207	518.2	1124246	528.4	1785518	521.5	1470530	519.2	1664108
103.5	110.4	417.7	1416158	424.5	2590764	427.1	1725302	418	1708906
69	207	416.8	1078980	418.6	1661772	419	1397828	414.2	1630164
69	138	342.1	1204514	351.5	1936552	351.8	1507636	344.8	1790510
69	82.8	288	1623222	292.1	1799738	293	1901072	291	1639364